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THE DESIGN OF

MINE STRUCTURES

BY

MILO S. KETCHUM, C.E.

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C. F.

PREFACE.

The aim in writing this book has been to present a systematic discussion of the design of mine structures. While the design of headworks for mines is given the principal place, the design of buildings, bins, coal washers and breakers, etc., is discussed as fully as the limited space permits. In the design of mine structures it is necessary that the structural engineer be familiar with the operation of mines and with various preparatory processes for ore and coal. The discussion of the operation of mines has been limited to the purpose of the structural engineer, and the design of hoists and other machinery has not been considered.

Many mine structures are statically indeterminate, so that it has been necessary to present a brief discussion of the calculation of stresses in statically indeterminate structures, and to show the application of the methods developed to the calculation of stresses in head frames. The methods for the calculation of stresses in simple framed structures, and the methods for calculating the pressures on retaining walls and the stresses in bins are briefly discussed.

A brief résumé of the design of reinforced concrete structures is given in Appendix III. Specifications are given for steel, timber and concrete mine structures, and data and details are given for use in design. The subject of costs has been given considerable attention and numerous examples of actual structures have been described in detail.

While this book is supplementary to the author's "The Design of Steel Mill Buildings" and "The Design of Walls, Bins and Grain Elevators," it is self contained.

This book is a result of the author's experience of several years in the design of mine structures and of a careful study of present practice. The book is written for the young engineer or student who has had a thorough course in applied mechanics and statics, and for the

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structural engineer who wishes to become familiar with this much neglected field.

As far as practicable credit has been given in the text of the book for all data and plans. The author is under obligations to many engineers for data and plans; he especially wishes to thank Mr. J. B. Gilman, Chief Engineer, The Minneapolis Steel and Machinery Company; Mr. Albert Reichmann, Division Engineer, American Bridge Company; Mr. C. W. Brooks, Contracting Engineer, Wisconsin Bridge and Iron Company; the Allen and Garcia Co.; and the engineers of the Wellman-Seaver-Morgan Company. Credit is due two of the author's former students, H. C. Ford, Associate Professor of Civil Engineering, Iowa State College, and W. C. Huntington, Instructor in Civil Engineering in the University of Colorado, for assistance in preparing the drawings.

M. S. K.

March 8, 1912.
UNIVERSITY OF COLORADO,
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THE DESIGN OF MINE STRUCTURES.

Introduction.—In addition to a knowledge of structural engineering the design of structures for mines requires a knowledge of the operations of hoisting from mines, and of the treatment and preparation of the ore or coal at the mine preliminary to shipment to the mill, smelter, or coking oven, for further treatment. In this book the discussion of mining and milling machinery is limited to a description of their operations in so far as the mine structures are affected thereby.

The book is divided into three parts and appendices containing specifications. The design of head works for mines is considered in Part I; the design of buildings, bins, retaining walls, coal washers and breakers, and miscellaneous structures in Part II; the general principles of design and the cost of mine structures in Part III; while specifications for steel mine structures are given in Appendix I, for timber mine structures in Appendix II, and for reinforced concrete structures in Appendix III.

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PART I.

THE DESIGN OF HEAD WORKS FOR MINES.

Introduction.—The head works for mines vary from very simple structures built of stock sizes of timber by an ordinary mechanic to elaborate structures in which the ore or coal is screened, crushed, sorted and prepared for further treatment or for market, and requiring the services of an experienced structural engineer.

This part of the book includes a discussion of different types of head works; the calculation of stresses in statically determinate and statically indeterminate head frames; and the design of head frames and coal tipples. The design of the steel frame buildings and bins that form parts of head works is discussed in Part II.

In writing Chapter III and Chapter IV, it has been assumed that the reader has had a thorough course in algebraic and graphic statics. In using this book as a text it is suggested that Chapter VIII be read before Chapter III; and that the author's "The Design of Steel Mill Buildings" and "The Design of Walls, Bins and Grain Elevators" be used as reference books.

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CHAPTER I.

Types of Head Works for Mines.

Introduction.—The design of the head works for a mine depends upon the material which is to be hoisted, upon the depth of the mine, the inclination of the shaft, the rate of hoisting, the amount to be hoisted at one time, the treatment of the ore or coal after being hoisted, and upon the material used in the construction of the structure. Head works for mines may be divided into three classes: (1) head frames; (2) rock houses; (3) coal tipples. Where coal, ore or rock is to be hoisted and taken directly away from the head works without further treatment the structure is called a head frame or "gallows frame." Coal is commonly screened after being hoisted and the head frame and the screening building together are called a coal tipple. Where ore or rock is to be crushed and sorted in the head works after it has been hoisted, the structure is called a rock house. The foregoing classification is not very definite, for in some cases the head frames are inclined and in others a small amount of screening is done, while in other cases head frames have bins and screens so that it is often difficult to determine whether the structure is a head frame or a rock house. Many of the coal tipples in the anthracite region of Pennsylvania are head frames from which the coal is conveyed to a coal breaker.

Head works for mines may be built of timber, steel or reinforced concrete. When timber was cheap and steel difficult to obtain head frames and coal tipples were built of timber. Many coal tipples have burned with a large property loss and loss of life so that timber coal tipples are now seldom used. The difficulty and cost of obtaining timber for large head frames has resulted in the building of many steel head frames for ore mines.

HEAD FRAMES.—The first head frames were constructed of timber; the most common type being the 4-post head frame shown in Fig. 1. The square or rectangular mine tower is cross-braced and the sheave supports are made of heavy timber. The back brace is inclined and is placed between the hoisting rope and the line of the resultant of the stress in the hoisting rope. This type of head frame gives fairly

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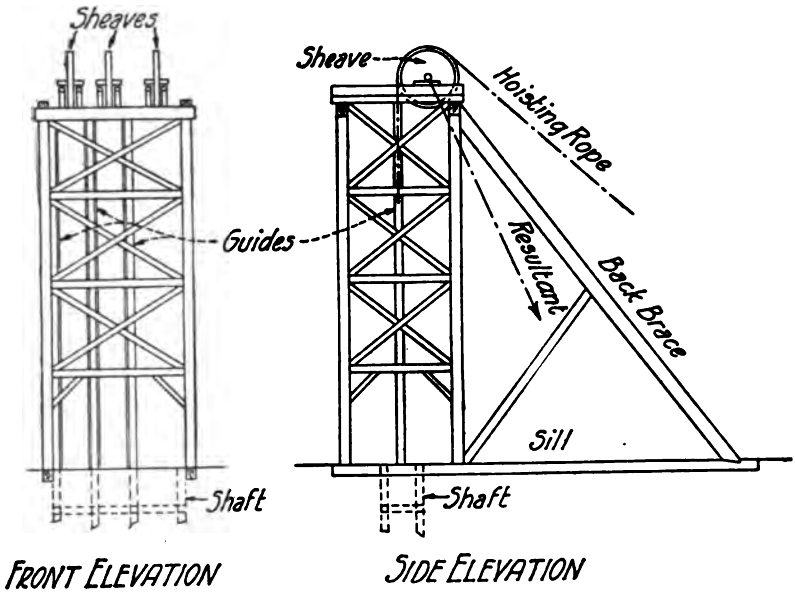

FIG. 1. TIMBER HEAD FRAME; 4-POST TYPE.

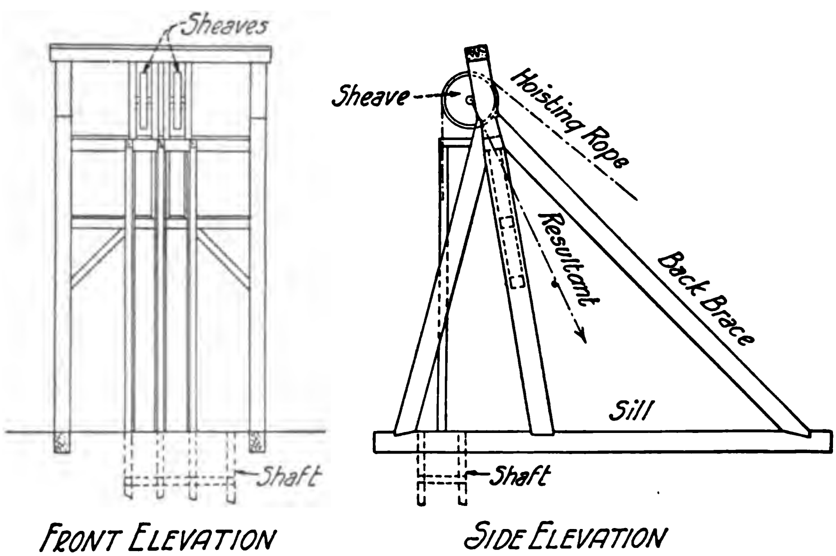

FIG. 2. TIMBER HEAD FRAME; MONTANA TYPE.

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satisfactory results but is expensive for high frames on account of the cost of obtaining the timber. The timber head frame shown in Fig. 2 was formerly used in Montana. The head frames at the Diamond and High Ore mines in Butte, Montana, were 100 feet from the bottom of the sill to the center of the sheaves.

While the Montana type of timber head frame appears to be a simpler and better designed frame than the 4-post frame in Fig. 1, the timbers are more expensive and the frame is not so well braced to with-

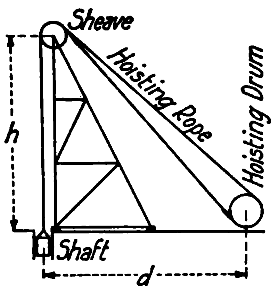

FIG. 3.

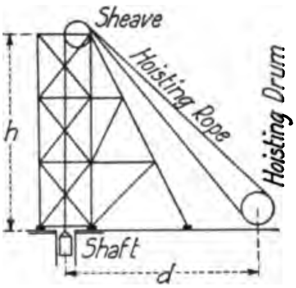

FIG. 4.

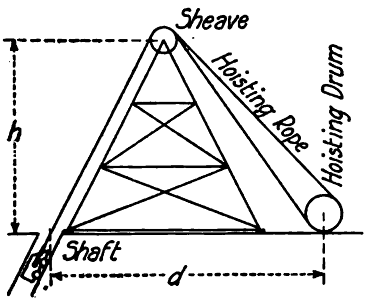

FIG. 5.

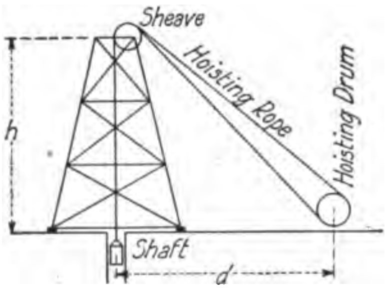

FIG. 6.

stand vibration. Most of the timber head frames have been of the 4-post type.

Steel head frames vary in design to suit local conditions and the ideas of the designer. The A-frame in Fig. 3 is the most satisfactory type where conditions permit of its use. It is simple in design and economical of material; the stresses are statically determinate, and it can be easily and effectively braced, making a very rigid frame. The 4-post frame in Fig. 4 is the type to use when it is necessary to hoist from several compartments of a shaft not in a single line. It is also

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Fig. 7. Steel Head Frame, Rand Consolidated Mines, Krugersdorp, Transvaal.

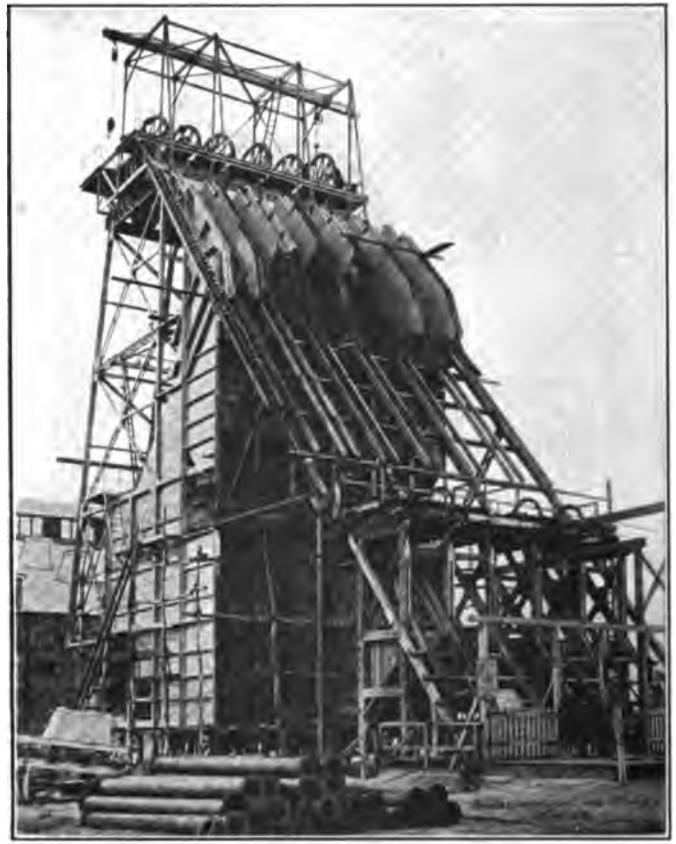

Fig. 8. Steel Head Frame, Rand Consolidated Mines, Krugersdorp, Transvaal.

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Fig. 9. Steel Head Frame, High Ore Mine, Butte, Mont. (For detail plans see Fig. 87.)

Fig. 10. Steel Head Frame, St. Lawrence Mine, Butte, Mont.; Built by the Wellman-Seaver-Morgan Company.

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used for coal tipples and for double compartment shafts. The 4-post frame is not so economical of material as the A-frame; is more difficult to brace effectively, partly for the reason that part of the bracing in the tower must be omitted to permit the dumping of the ore or coal, and in addition the stresses are statically indeterminate. The frame shown in Fig. 5 is a modification of the A-frame used for an inclined shaft.

FIG. 11. STEEL ROCK HOUSE FOR QUINCY MINE NO. 2, HANCOCK, MICH.; BUILT BY AMERICAN BRIDGE COMPANY.

Several early head frames in the coal fields of Pennsylvania were built on the lines of the frame shown in Fig. 6. This type of frame has no points of merit and is practically obsolete.

One of the earliest if not the earliest steel head frame in the United States was built in 1870 at the Oakwood shaft of the Lehigh Valley Coal Company. It is 55 feet high and is built of wrought iron channel bars, laced. It has its posts nearly vertical, Fig. 6, and is elaborately braced, but has no back braces. Steel head frames were used in the early development of mines in South Africa. The steel head frame at

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the West Rand Consolidated Mines near Krugersdorp in the Transvaal, is shown in Fig. 7 and Fig. 8 This head frame is 92 feet from the collar of the shaft to the top of the sheaves. There is an ore bin with a capacity of 1,000 tons, a waste bin with a capacity of 150 tons, and a water tank with a capacity of 6,000 gallons. The mine has a six compartment shaft as is shown in Fig. 8.

FIG. 12. FRAMEWORK FOR STEEL COAL TIPPLE; PLAIN CAGE HOISTING.

The timber head frame in Fig. 2 at the High Ore mine was replaced by the steel head frame in Fig. 9. This head frame is 100 feet high from the collar of the shaft to the center of the sheaves. The plans of this head frame are shown in Chapter VI, Fig. 87.

The steel head frame for the St. Lawrence mine shown in Fig. 10, was built by the Wellman-Seaver-Morgan Company in 1897, and was the second steel head frame built in the Butte district. The frame is 97 feet high from the collar of the shaft to the center of the sheaves, which are 10 ft. in diameter. The hoisting rope is 7 in. × $\frac{1}{2}$ in. The

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ore is hoisted in self-dumping 7-ton skips weighing $3\frac{1}{2}$ tons. Additional data on this head frame are given in Table LIII, Chapter XV.

ROCK HOUSES.—The steel rock house of the Quincy, No. 2 mine, is shown in Fig. 11. The steel head frame is 119′ 3″ high from the collar of the shaft to the center of the sheaves. The shaft is inclined at an angle of approximately 57° with the horizontal. The rock is of the amygdaloid formation and contains pure native copper, varying in size from fine grains to large masses. In the rock house the poor rock is separated; the pay rock is crushed to proper size, and the mass copper is separated from the rock clinging to it by placing it under a steam hammer. This rock house is therefore a large crushing and sorting plant. Detail plans of the rock house are shown in Fig. 109, and a description is given in Chapter VI.

COAL TIPPLES.—The design of a coal tipple depends upon the inclination of the shaft, the method of hoisting, the work to be done

FIG. 13. FRAMEWORK FOR STEEL COAL TIPPLE; SELF-DUMPING CAGES.

on the coal, and upon the arrangement of the screens. The shaking screens which are commonly used in coal tipples produce excessive vibrations unless the shaker building is very thoroughly braced. The

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FIG. 14. STEEL COAL TIPPLE, CARNEY COAL COMPANY, CARNEYVILLE, WYO.; BUILT BY WISCONSIN BRIDGE AND IRON COMPANY.

latest practice in the design of coal tipples is to build the head frame and the shaker building as separate, independent structures. The frame work for a steel coal tipple in Central Illinois is shown in Fig. 12. The

FIG. 15. STEEL COAL TIPPLE, LORAIN COAL AND DOCK CO.; BUILT BY JEFFREY MANUFACTURING COMPANY.

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sheaves of the head frame are placed parallel with the hoisting rope and parallel to the axis of the coal tipple. Coal tipples are also built with the hoisting drum at one side of the tipple building, with the sheaves in tandem as shown in Fig. 127. The coal is hoisted in cars which are dumped into weigh boxes, from which the coal is run over screens into railroad cars placed underneath the structure.

FIG. 16. FILBERT COAL TIPPLE DURING ERECTION.

The frame work of a steel tipple building in which self-dumping cages are used is shown in Fig. 13. The sheaves of the head frame are placed parallel with the hoisting rope and at right angles to the railroad tracks. After the coal is hoisted it is dumped into weigh boxes, and after running over the screens is spouted into railroad cars placed beneath the structure.

It is sometimes necessary to sort bituminous coal, in which case part

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of the coal is run both over sorting tables and over shaking screens. For a description of the coal tipple of the Alberta Railway & Irrigation Company see Chapter VII and Fig. 136.

Where coal is taken from a tunnel or a drift it is necessary to design a different type of coal tipple. The steel coal tipple for the Carney Coal Company at Carneyville, Wyoming, is shown in Fig. 14. In this coal tipple the coal is carried from the mine by an inclined con-

Fig. 17. Filbert Coal Tipple Completed.

veyor with a capacity of 500 tons per hour. The tipple and screens are so arranged that lump and egg coal can be loaded into box cars at the same time, the screens being placed parallel to the railroad tracks. The capacity of the screens and loading equipment is 2,500 tons in eight hours. A steel coal tipple for a mine in which the coal is brought from a mine tunnel on cars is shown in Fig. 15. A reinforced concrete coal tipple is now being constructed by the Leyden Coal Company, Leyden, Colorado.

The steel coal tipple for the Filbert mine of the H. C. Frick Coke Co. is shown during erection in Fig. 16, and after completion in Fig. 17. The coal is hoisted in self-dumping cages, and is run into the bins

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without screening. It is then conveyed to the coke ovens in electric cars, called larries. The plans for the steel tipple at the Phillips mine of the H. C. Frick Coke Co. are shown in Fig. 143, Chapter VII.

A tipple for an anthracite coal mine differs from a tipple for a bituminous coal mine in that anthracite coal is commonly conveyed from the head frame to a coal breaker, where it is broken up into various commercial sizes and prepared for the market.

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CHAPTER II.

Hoisting from Mines.

METHODS OF HOISTING.—In hoisting from inclined or vertical shafts, the hoisting engine is placed at some distance from the mouth of the shaft, the cable passes up over the sheave at the top of the head frame and into the shaft. The rope, if round, is carried on a

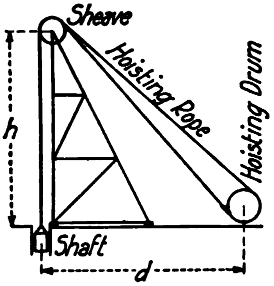

Fig. 18.

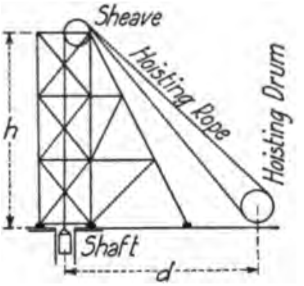

Fig. 19.

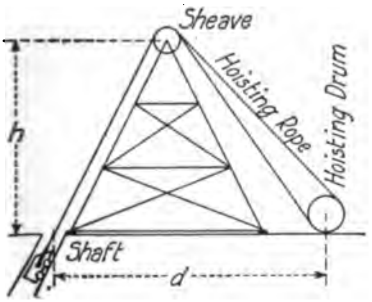

Fig. 20.

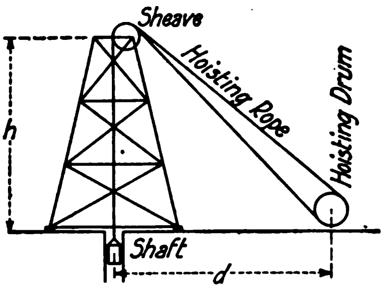

Fig. 21.

smooth or a grooved hoisting drum, and if flat, is carried on a hoisting reel. The maximum working load on the rope occurs when the loaded skip or cage is being hoisted from the bottom of the shaft. The working load then consists of the skip or cage, the load, the accelerating force, the weight of the rope itself, and the friction of the rope on the sheave and drum and of the skip or cage in the guides.

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With round ropes the hoisting drum for deep mines is commonly made conical, the small diameter being used when the load is at the bottom of the shaft. Flat ropes are wound on a reel, so that the small diameter is used when the load is at the bottom of the shaft, the diameter of the reel increasing as the rope is wound up. The required height of the head frame depends upon (1) the room required for screening, crushing and handling the coal or ore; (2) the speed of hoisting—with rapid hoisting it is necessary to have a height from the

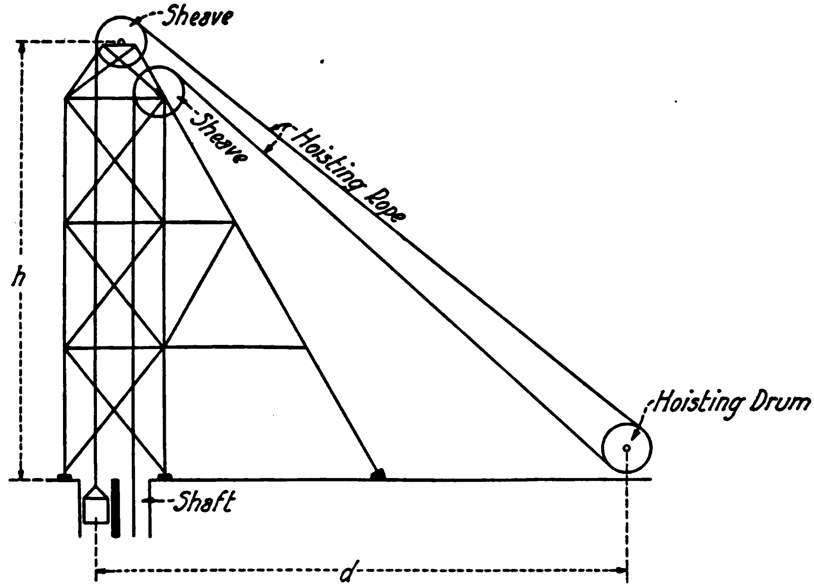

FIG. 22.

landing to the sheaves of from two to three times the height of the cage or skip or a full revolution of the drum to prevent over winding, and (3) the desired location of the hoisting engine. With a given height of head frame *h*, the distance *d*, Figs. 18 to 22, depends upon the diameter of the sheave, the diameter of the rope, and whether the rope is round or flat. The sheave should be as large as can conveniently be used, as the larger the sheave the longer the life of the hoisting rope. The inertia of a large, heavy sheave, however, with rapid hoisting may kink the rope and cause excessive wear. The bending stresses in flat ropes for a sheave of given diameter are less than in round ropes having equal strength, but the life of flat ropes is less than for round

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ropes. Flat ropes are wound on reels which are at all times in line with the head frame sheave, while round ropes are wound on a drum so that the horizontal angle between the center line of the sheave and the cable is continually changing. The distance, *d*, for flat ropes can then be less than for round ropes.

Hoisting from mine shafts is commonly done in two compartments of the shaft at the same time, the unloaded skip or cage descending as the loaded skip or cage ascends. This is known as hoisting in balance or counterbalance. There is a considerable saving in power in hoisting in balance. To hoist in balance it is necessary to take ore from one ' vel with both skips unless the Whiting system is used. When a

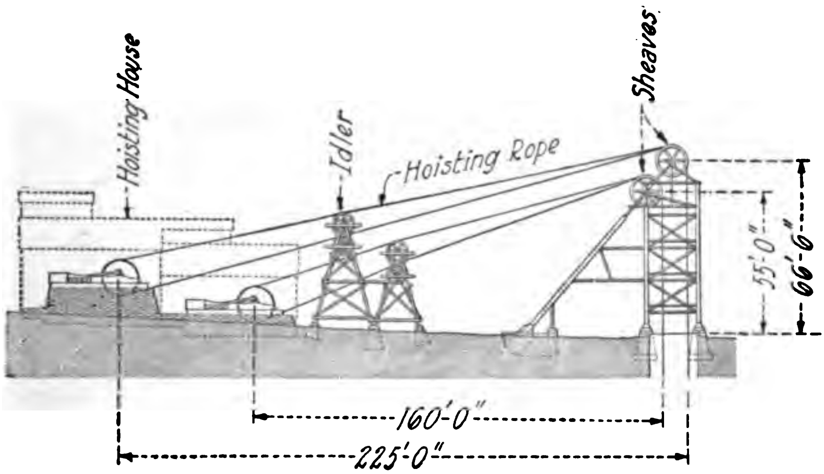

FIG. 23. GILBERTON STEEL HEAD FRAME.

round rope winds off the drum it makes an angle with the groove in the sheave on the head frame and the friction increases the tension in the cable and also reduces its life. To reduce the friction and wear the hoisting engines are placed at a considerable distance back from the head frame.

The head frame may be placed so that the sheaves are parallel, as in Figs. 18 to 21, or so that the sheaves are in tandem, as in Figs. 22 and 23. With the latter method it is necessary to place the hoisting engine farther from the shaft than where the sheaves are in parallel. Where the hoisting engine is placed well back from the shaft it becomes necessary to support the hoisting rope on idlers, as shown in Fig. 23. Where

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mines have three compartment shafts, ore is commonly hoisted from but two compartments, the third compartment being used for pumps, pipes, etc. This arrangement makes it necessary to place the head sheaves so that they will not be symmetrical with the center line, bringing heavier working stresses on one side of the head frame than on the other side.

HOISTING FROM DEEP MINES.—In deep mines the rope in the mine becomes a large part of the load and various methods have

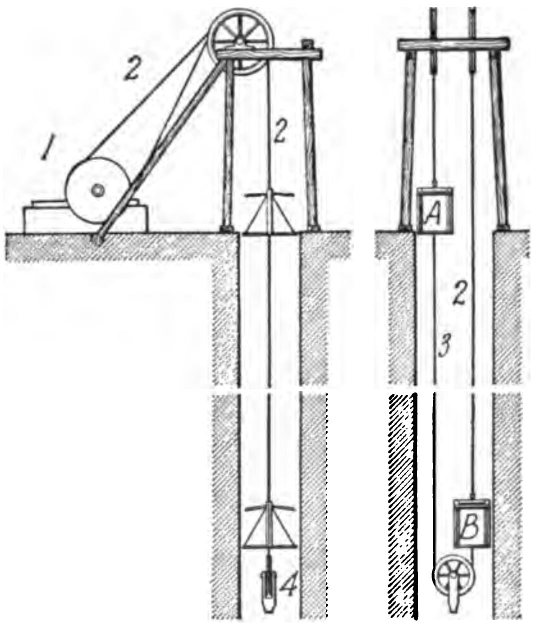

FIG. 24. KOEPE SYSTEM.

been used to counterbalance the weight of the rope. Four methods for obviating the difficulty just mentioned have been used: (1) the Koepe system; (2) the Whiting system; (3) modifications of (1) and (2), and (4) by the use of a taper rope.

The Koepe System.—In the Koepe system, Fig. 24, a large grooved wheel, *1*, is placed on the engine crank shaft. The rope is passed half around this wheel, *1*, and over two sheaves at the head frame, and then to the top of the cages—one cage being at the top landing and the

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other at the bottom of the shaft. A balance rope, *3*, is fastened to the under side of each cage. The driving wheel, *1*, must be made large to give sufficient frictional resistance. To prevent slipping it is necessary to start the load very slowly. The sheaves on the head frame may be parallel or tandem, the latter method being preferred. A modification of the Koepe system is to use two driving reels on pulleys on the engine shaft, and a transfer wheel placed between the hoisting engine and the head sheave. The rope from the head sheave passes around the first driving wheel on the engine shaft, then around the transfer wheel, then around the second driving wheel on the engine shaft, and then to the second head sheave. The Koepe system has the following disadvantages. (1) In case the hoisting rope breaks both cages fall down the shaft; (2) it is difficult to socket the hoisting rope; and (3) the extra stress on the rope fastenings due to the weight of the balance rope. The system is adapted only for deep hoisting where the slow speed in starting is not a serious handicap. A speed of 30 ft. per second has been reached in hoisting from a shaft 1,300 feet deep. The Koepe system is used in Germany and England, and in a modified form in the United States.

Whiting System.—In the Whiting system, Fig. 25, the rope starts from the top of the cage, *A*, goes over the head sheave, *2*, down to the

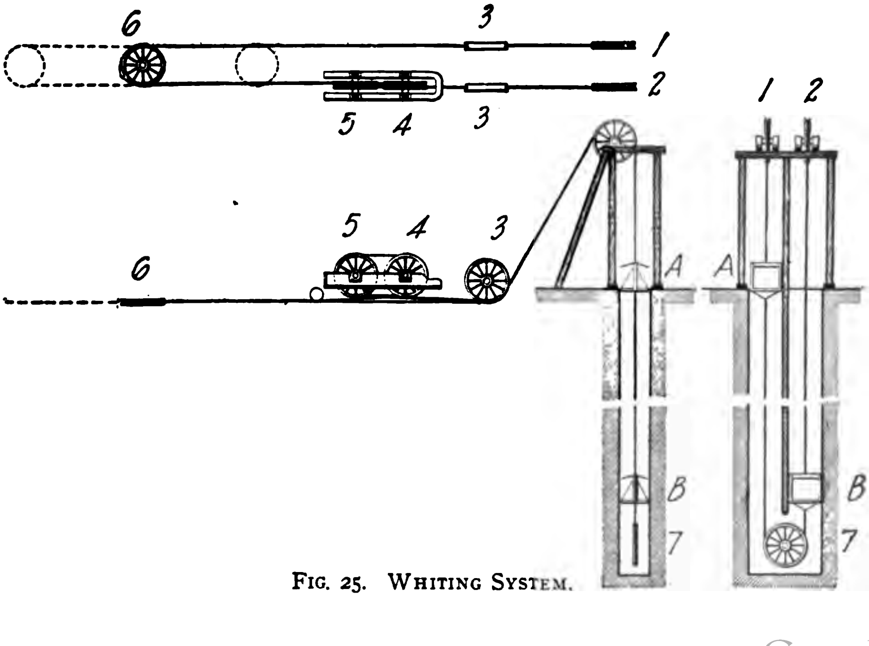

FIG. 25. WHITING SYSTEM.

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guide wheel, *3*, from underneath which it goes to the underside of the drum *4*, where it makes one half turn to drum *5*. The rope is given three half turns over this drum, and passing off from underneath drum *5*, goes to tension car *6*, then half around this, back to and under another guide pulley up to the second head sheave and down to cage *B*. Drums *4* and *5* have their shafts slightly inclined so that the rope will run from one drum to the other without friction or binding. The object of the tension car *6* is to permit hoisting from different levels, the dotted circles showing its position for different levels. When hoisting from different levels at the same time it is necessary to change the tension car quickly, the tension car being clamped down during hoisting. A tail rope is used for a counterbalance. During sinking operations tail ropes cannot be used. The Whiting system is used principally in the upper Peninsula of Michigan and is a decided improvement on the Koepe system.

Taper Ropes.—The stresses in a hoisting rope are due to the weight of the cage and the load, impact due to sudden starting, and the weight of the rope itself. A hoisting rope of equal strength in all parts should therefore be tapered, having a minimum section at the lower end. Taper ropes are usually made by dropping out wires at intervals. Taper round hoisting ropes have been used to a limited extent in hoisting from deep mines. Taper flat ropes do not give satisfactory results for the reason that the rope is liable to wedge in the reel with serious results.

HOISTING ROPES.—Round hoisting ropes are commonly made of six strands, each of which is formed by twisting nineteen wires together, the strands being wound around a hemp center. Wire strands are twisted around the core either to the right or the left, and the resulting rope is either "right lay" or "left lay." The twist may be long or short; the shorter twist forms a more flexible rope, while the longer twist forms a more rigid rope. Wire rope is made of iron, open hearth steel, crucible steel, and plough steel. The strength of the wire from which the rope is made is about as follows: iron wire, 40,000 to 100,000 lbs. per sq. in.; open hearth steel wire, 50,000 to 130,000 lbs. per sq. in.; crucible steel wire, 130,000 to 190,000 lbs. per sq. in.; and plough steel wire, 190,000 to 350,000 lbs. per sq. in. Hoisting ropes are usually made of crucible cast steel or plough steel.

Flat wire rope is composed of several round ropes whose diameter

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is equal to the required thickness of the flat rope, laid side by side and sewed together with iron or annealed cast steel wire. The round ropes are alternately of right and left lay or twist, and have four strands without either hemp or wire center. The number of wires in each strand is usually seven, but may be nineteen. The chief drawbacks to the use of flat wire rope are its first cost and the rapid wear of the sewing wires.

Flat ropes and reels are used to a limited extent in the western part of the United States, while round ropes are generally used in hoisting coal and in the deep copper and iron mines in Michigan.

Strength of Wire Rope.—The dimensions, weight and strength of round crucible steel hoisting rope are given in Table I, while similar data for plough steel hoisting rope are given in Table II. The strengths of wire rope given by the different makers differ somewhat.

TABLE I.

CAST STEEL HOISTING ROPE. ULTIMATE STRENGTH, WORKING STRENGTH AND WEIGHT OF WIRE ROPE COMPOSED OF 6 STRANDS AND A HEMP CENTER, 19 WIRES TO THE STRAND.

Diameter, In.	Approximate Circumference, In.	Weight per Ft., Lbs.	Safe Working Load, for Hoisting, L, Lbs.	Approximate Breaking Stress, Lbs.	Safe Working Stress, for Direct Pull, S, Lbs.	Minimum Size of Drum or Sheave, Ft.
2¾	8⅝	11.95	Safe working load, $L, = 2S$—bending stress.	456,000	76,000	10
2½	7⅞	9.85		380,000	66,300	9½
2¼	7⅛	8.00		312,000	52,000	8½
2	6¼	6.30		248,000	41,300	8
1¾	5½	4.85		192,000	32,000	7¼
1⅝	5	4.15		168,000	28,000	6¼
1½	4¾	3.55		144,000	24,000	5¾
1⅜	4¼	3.00		124,000	20,700	5½
1¼	4	2.45		100,000	16,700	5
1⅛	3½	2.00		84,000	14,000	4½
1	3	1.58		68,000	11,300	4
⅞	2¾	1.20		52,000	8,700	3½
¾	2¼	0.89		38,800	6,300	3
⅝	2	0.62		27,200	4,500	2¼
9/16	1¾	0.50		22,000	3,700	1¾
½	1½	0.39		17,600	2,900	1½
7/16	1¼	0.30		13,600	2.300	1¼
⅜	1⅛	0.22		10,000	1,670	1
5/16	1	0.15		6,800	1,170	⅔
¼	¾	0.10		4,800	800	½

The strengths of flat cast steel ropes are given in Table III.

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TABLE II.

PLOUGH STEEL HOISTING ROPE. ULTIMATE STRENGTH, WORKING STRENGTH AND WEIGHT OF WIRE ROPE COMPOSED OF 6 STRANDS AND A HEMP CENTER, 19 WIRES TO THE STRAND.

Diameter, In.	Approximate Circumference, In.	Weight per Ft., Lbs.	Safe Working Load for Hoisting, L, Lbs.	Approximate Breaking Stress, Lbs.	Safe Working Stress for Direct Pull, S, Lbs.	Minimum Size of Drum or Sheave, Ft
2 3/4	8 5/8	11.95	Safe working load, L, $= 2S$ — bending stress.	550,000	91,700	14
2 1/2	7 7/8	9.85		458,000	76,300	12 1/2
2 1/4	7 1/8	8.00		372,000	62,000	11
2	6 1/4	6.30		280,000	47,700	9 3/4
1 3/4	5 1/2	4.85		224,000	37,300	8 1/2
1 5/8	5	4.15		188,000	31,300	7 3/4
1 1/2	4 3/4	3.55		164,000	27,300	7
1 3/8	4 1/4	3.00		144,000	24,000	6 3/4
1 1/4	4	2.45		116,000	19,300	6
1 1/8	3 1/2	2.00		94,000	15,700	5
1	3	1.58		76,000	12,700	4 1/2
7/8	2 3/4	1.20		58,000	9,700	4
3/4	2 1/4	0.89		46,000	7,700	3 1/4
5/8	2	0.62		31,000	5,170	2 3/4
9/16	1 3/4	0.50		24,600	4,100	2 1/8
1/2	1 1/2	0.39		20,000	3,300	2
7/16	1 1/4	0.30		16,000	2,700	1 3/4
3/8	1 1/8	0.22		11,500	1,900	1 1/2
5/16	1	0.15		7,600	1,270	1 1/8
1/4	3/4	0.10		5,300	890	1

TABLE III.

CAST STEEL FLAT HOISTING ROPE. ULTIMATE STRENGTH, WORKING STRESS AND WEIGHT OF FLAT WIRE ROPE COMPOSED OF 4 STRANDS, 7 WIRES TO THE STRAND.

Width and Thickness, In.	Weight in Lbs. per Lineal Foot.	Safe Working Load for Hoisting, L, Lbs.	Approximate Breaking Stress, Lbs.	Safe Working Stress for Direct Pull, S, Lbs.	Approximate Diameter in Inches of Round Cast Steel Rope of Equal Strength.
3/8 × 5 1/2	3.90	Safe working load, L, $= 2S$ — bending stress.	110,000	18,300	1 5/16
3/8 × 5	3.40		100,000	16,700	1 1/4
3/8 × 4 1/2	3.12		94,000	15,700	1 3/16
3/8 × 4	2.86		86,000	14,300	1 1/8
3/8 × 3 1/2	2.50		76,000	12,700	1
3/8 × 3	2.00		60,000	10,000	15/16
3/8 × 2 1/2	1.86		56,000	9,300	7/8
3/8 × 2	1.19		36,000	6,000	3/4
1/2 × 7	5.90		178,000	29,700	1 5/8
1/2 × 6	5.10		154,000	25,700	1 9/16
1/2 × 5 1/2	4.82		144,000	24,000	1 1/2
1/2 × 5	4.27		128,000	21,300	1 3/8
1/2 × 4 1/2	4.00		120,000	20,000	1 5/16
1/2 × 4	3.30		100,000	16,700	1 1/4
1/2 × 3 1/2	2.97		90,000	15,000	1 1/8
1/2 × 3	2.38		72,000	12,000	1

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Working Load on Hoisting Rope.—The stresses in a hoisting rope are the sum of the stresses due to (1) the weight of the rope, (2) the friction of the rope, (3) the bending of the rope over the head sheave, (4) the live load, and (5) the impact due to starting and stopping the load. The stresses due to bending are discussed in the next section. The stresses due to impact vary from zero to twice the working load if the hoisting cable is taut, and to several times the working load if the cable is slack. If a descending cage should stick and then drop, the stress will be equal to the kinetic energy developed and will be very large. The load due to starting a cage suddenly from the bottom of a shaft may be taken as

$$K = 2W + R + F \qquad (1)$$

where K = stress in lbs. at the sheave at the instant of picking up the load;
W = gross load in lbs.;
R = weight of rope in lbs.;
F = friction in lbs., $= (W + R)f$, where f = coefficient of friction, which may be taken at 0.01 to 0.02 for vertical shafts and from 0.02 to 0.04 for inclined shafts with the rope supported on rollers. The working load should not be greater than K plus the stress due to bending, and should not exceed $\frac{1}{3}$ of the ultimate strength of the rope, or $\frac{1}{8}$ of the ultimate strength for direct pull.

For inclined shafts with angle of inclination with horizontal $= \theta$, the stress in the rope due to starting the cage is

$$K' = (2W + R) \sin\theta + f(W + R) \cos\theta \qquad (2)$$

Bending Stresses in Wire Rope.—The stresses due to bending will depend upon the diameter of the rope, the make-up of the rope, the angle through which the rope is bent, and the diameter of the sheave. The unit stress due to bending in a round hoisting rope may be obtained from formula (3), the form of which is due to Rankine ("Machinery and Mill Work," p. 533).

$$S = 1,894,000 \frac{d}{D} \qquad (3)$$

where D = the diameter of the sheaves in inches, and d = the diameter of the rope in inches. The area of the steel in a round hoisting rope is approximately, $a = 0.4 d^2$, and the total bending stress in a round rope will be

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$$S_b = S \cdot a = 757{,}600\frac{d^3}{D} \tag{4}$$

Now the direct breaking strength of a crucible steel round rope is closely

$$U = 60{,}000\,d^2 \tag{5}$$

Where bending stress is considered, the safe working load should not exceed $\frac{1}{3}$ of the ultimate strength, and the safe working load, L, should not exceed

$$L = 20{,}000d^2 - 757{,}600\frac{d^3}{D} \tag{6}$$

The safe working loads for crucible steel round ropes based on formula (6) are given in Fig. 26.*

For plough steel ropes the ultimate strength is $U = 70{,}000\,d^2$, and

$$L = 26{,}700d^2 - 757{,}600\frac{d^3}{D} \tag{6'}$$

Mr. William Hewitt in "Wire Rope," published by the Trenton Iron Company, gives the following formula for bending.†

$$S_b = \frac{E \cdot a}{1.03\frac{D}{d'} + C} \tag{7}$$

where E = the modulus of elasticity of steel, a = the area of the rope in sq. in., D = the diameter of the sheave in inches, d' = the diameter of the individual wires in inches, and C = a constant depending upon the rope, and varies from 9.27 for haulage rope to 27.81 for tiller rope. For standard hoisting rope, $C = 15.45$. Substituting $E = 29{,}000{,}000$, $a = 0.4\,d^2$, and $d' = \frac{d}{15}$, we have

$$S_b = \frac{750{,}000d^3}{D - d} \tag{8}$$

Since d is very small as compared with the values of D used in hoisting, formulas (4) and (8) give practically the same results.

* Redrawn from a diagram prepared by Mr. E. T. Sederholm, Chief Engineer Allis-Chalmers Company.

† Also see Engineering News, May 7, 1896.

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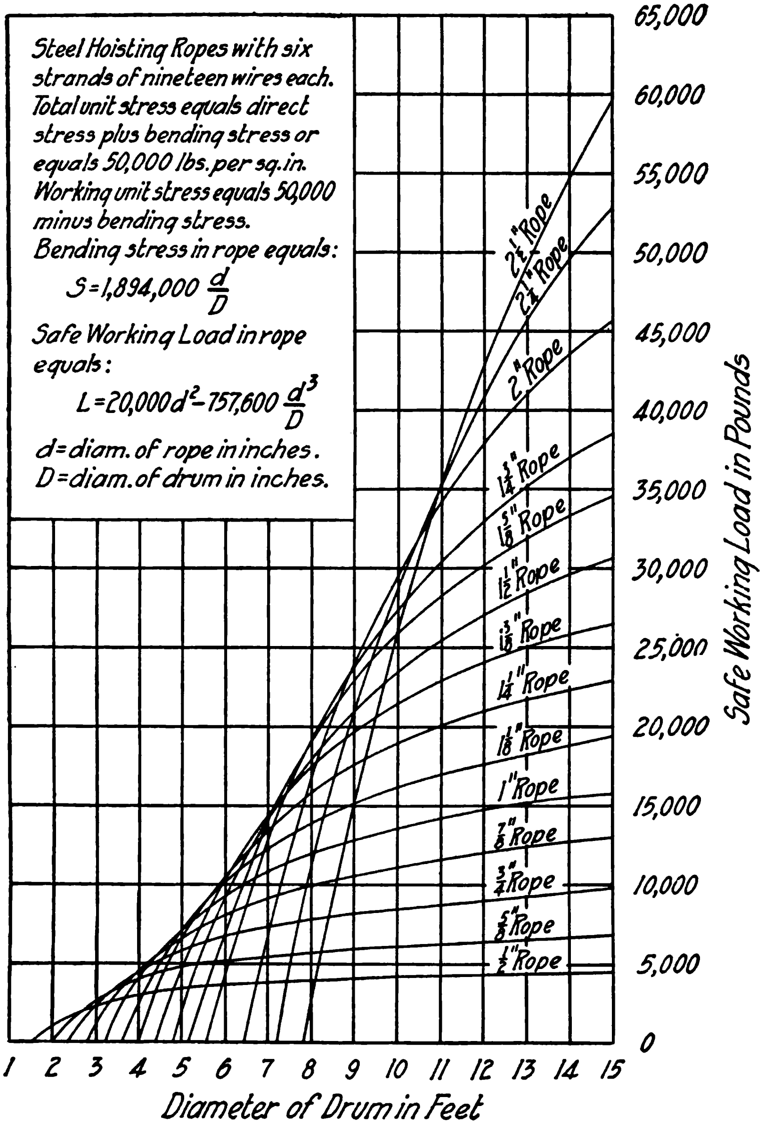

FIG. 26. SAFE WORKING STRESSES, L, IN CRUCIBLE STEEL, ROUND HOISTING ROPE.

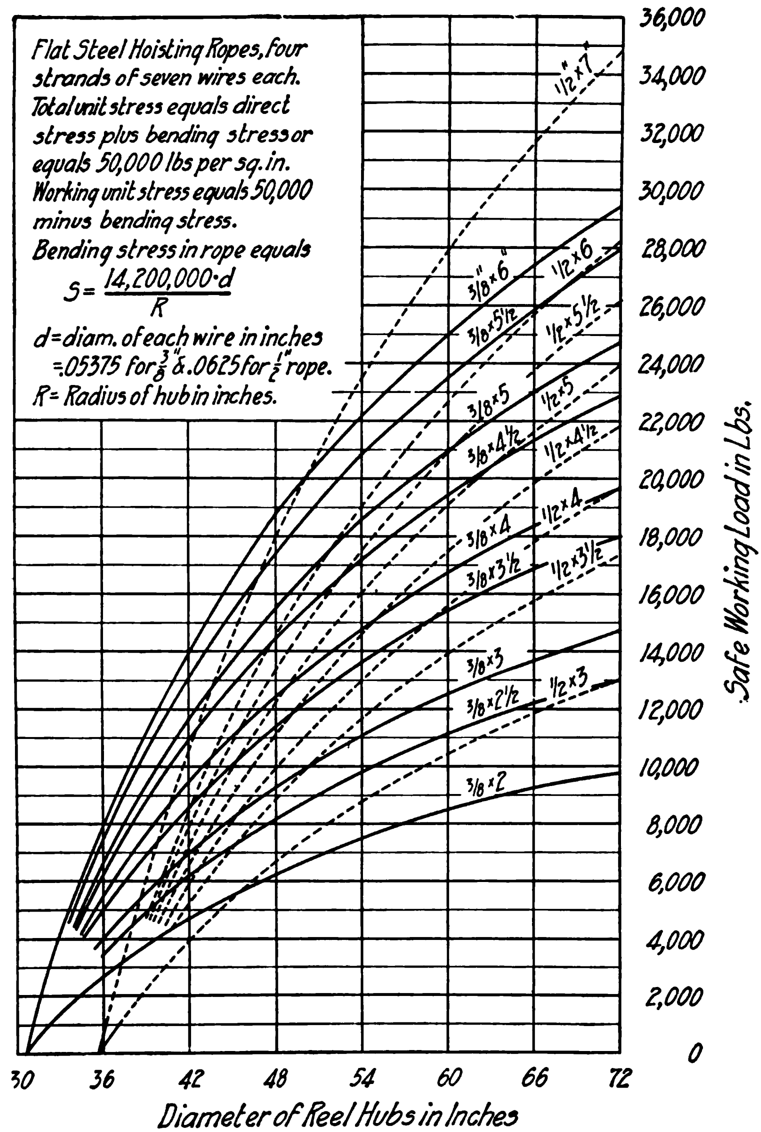

FIG. 27. SAFE WORKING STRESSES, L, IN CRUCIBLE STEEL, FLAT HOISTING ROPE.

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The bending stresses in crucible steel flat ropes are given in Fig. 27.*

Limit of Vertical Hoisting.—The present maximum depth of vertical hoisting is about 5,300 ft.; the deepest shaft being at the Tamarack mine in Michigan. With hoisting ropes of constant cross-section the practical limit of hoisting is about 6,000 ft. In a paper on "Winding Plants for Great Depths,"† read before the Institution of Mining and Metallurgy and also before the South African Association of Engineers, Mr. Hans C. Behr concludes that for depths of more than 6,000 ft. double stage hoisting will be necessary. In a paper read before the Colorado Scientific Society, July 2, 1904, Mr. Charles W. Comstock shows that with a taper hoisting rope it is entirely practicable to hoist

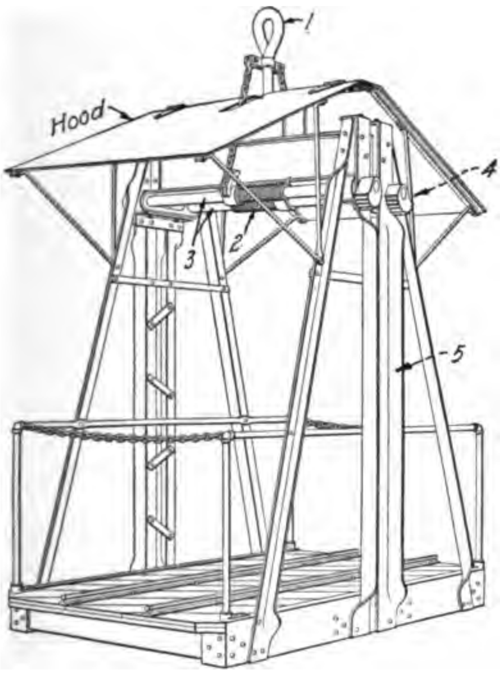

FIG. 28. SINGLE DECK CAGE.

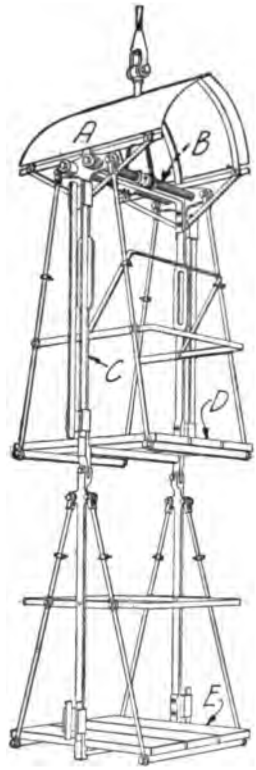

FIG. 29. DOUBLE DECK CAGE.

* Redrawn from a diagram prepared by Mr. E. T. Sederholm, Chief Engineer Allis-Chalmers Company.

† Transactions Institution of Mining and Metallurgy, Vol. XI, 1902.

from a vertical shaft 12,000 ft. deep in a single stage. A taper round rope was to be used with a conical hoisting drum.

CAGES.—Where the material is hoisted in mine cars, a cage is used for hoisting. Cages may be made with a single deck or with two, three or four decks. A single deck cage is shown in Fig. 28. Safety catches are shown at 4, Fig. 28. The hoisting rope is fastened to clevis, 1, which is fastened to a rod, 3, which carries a spiral spring, 2. Rod

FIG. 30 a. WELLMAN-SEAVER-MORGAN COMPANY'S LIGHT STEEL CAGE.

3 is fastened by levers to the safety catches, 4. The safety catches are thrown into action by the breaking of the hoisting rope or by striking the cage on the head sheave in overwinding. Safety hooks may also be used, see Fig. 30 a and Fig. 40. A double deck steel cage is shown in Fig. 29. A light steel cage manufactured by the Wellman-Seaver-Morgan Company is shown in Fig. 30 a. The dimensions of the steel cage are: platform 3 ft. × 5 ft.; distance between guides 3 ft. 4 in.; guides 4 in. × 6 in.; safe load 4,000 lbs.; weight 1,200 lbs. The above company makes heavy steel cages with the following limits of dimensions: platform, 4.25 ft. × 6 ft. to 6 ft. × 10 ft.; guides, 6 in. × 8 in. to 8 in. × 10 in.; distance between guides, 4.5 ft. to 6.25 ft.; safe load, 5,000 lbs. to 8,000 lbs.; weight of cage, 2,000 lbs. to 3,800 lbs.

Landers.—When the cage reaches the surface or the level of the landing floor a device called a lander is required to prevent the cage from dropping back into the shaft. Landers usually act automatically after the cage has been hoisted up and must be released before the

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cage can descend. A steel lander for the Annabelle Mine of the Four States Coal & Coke Co. is shown in Fig. 30 b. Also see Fig. 144 e, Chapter VII.

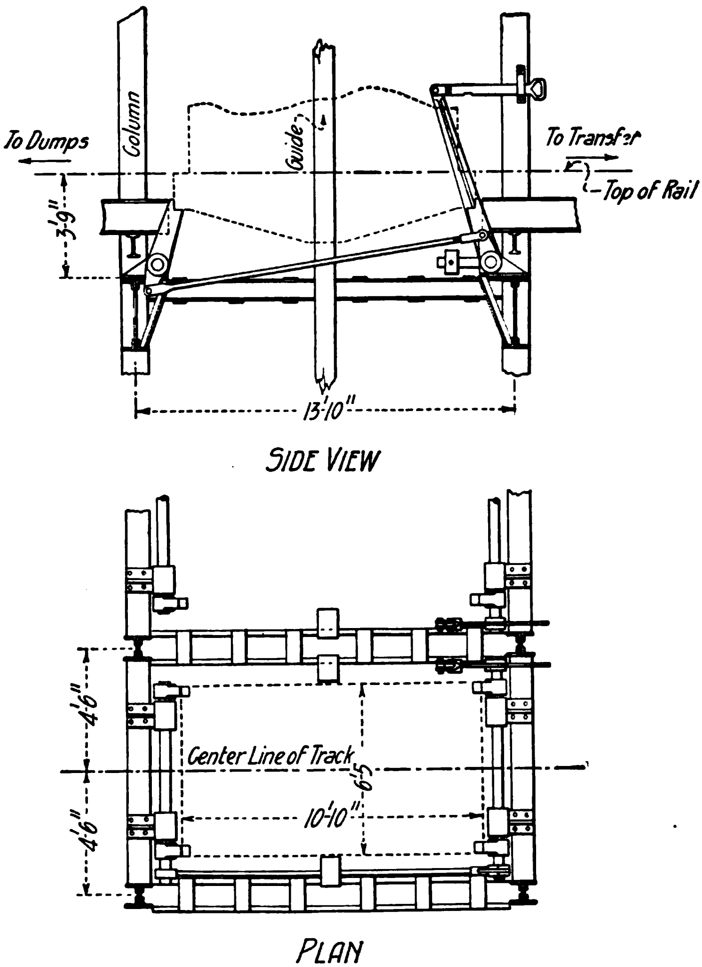

FIG. 30 b. STEEL LANDER FOR ANNABELLE MINE.

Details of the three deck cage used by the Copper Queen Mining Company are shown in Fig. 92 and Fig. 93, Chapter VI, while details of cages used at the Sibley and Savoy mines are shown in Fig. 104 and Fig. 105, respectively.

A steel cage for an inclined shaft is shown in Fig. 31; skips are commonly used in inclined shafts. A self dumping cage is shown in

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FIG. 31. STEEL CAGE FOR AN INCLINED SHAFT.

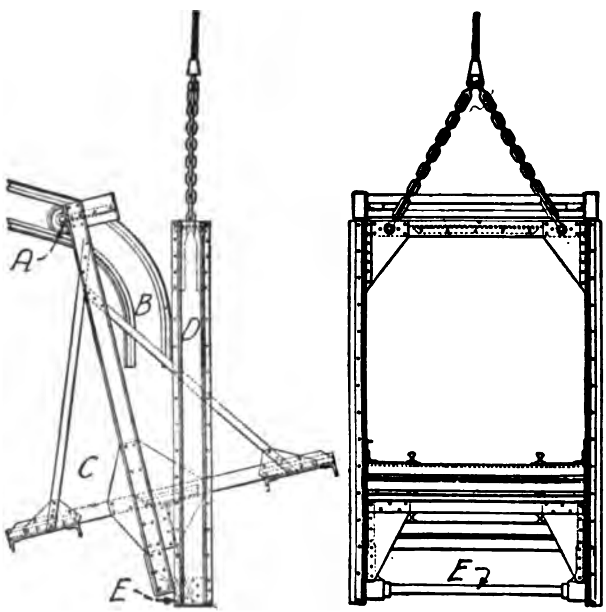

FIG. 32. SELF DUMPING CAGE.

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Fig. 32. Self dumping cages are used principally in coal mines where the extra handling made necessary by the use of skips is objectionable. Details of the self dumping cage used at the Phillips mine are shown in Fig. 144.

SKIPS.—Skips are commonly used in inclined shafts, and are used in vertical shafts where a large amount of ore is to be hoisted. Skips

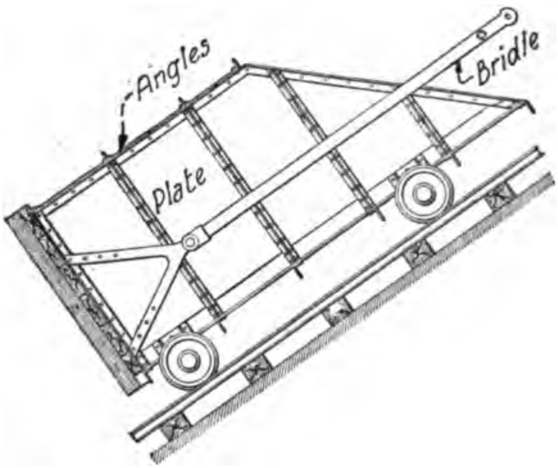

Fig. 33. Mine Skip for an Inclined Shaft.

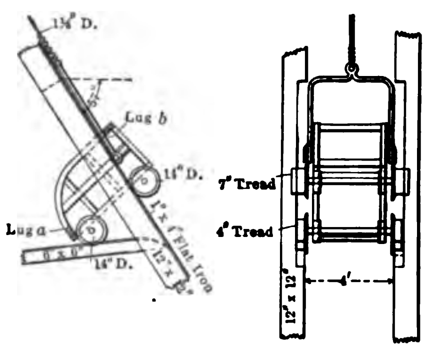

Fig. 34. Skip for Inclined Shaft.

are rarely used in hoisting bituminous coal for the reason that their use makes an additional handling of the coal necessary in the mine. A skip for an inclined shaft is given in Fig. 33.

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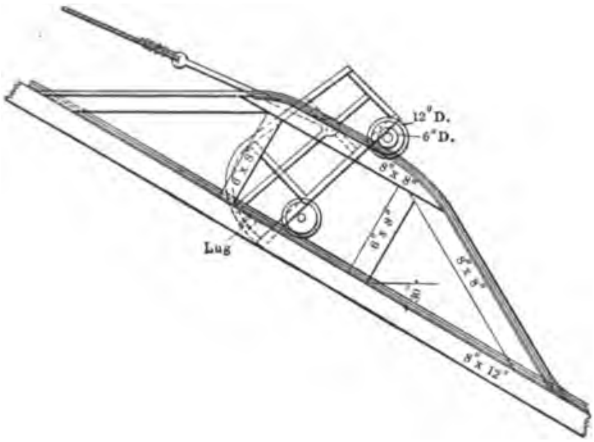

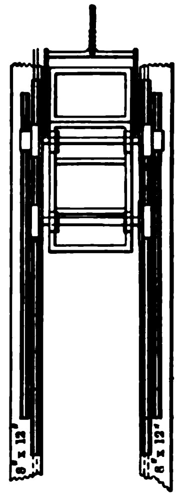

FIG. 35. SKIP FOR INCLINED SHAFT.

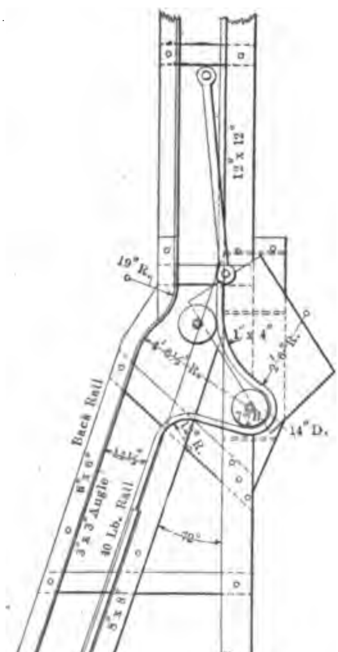

FIG. 36. AUTOMATIC SKIP.

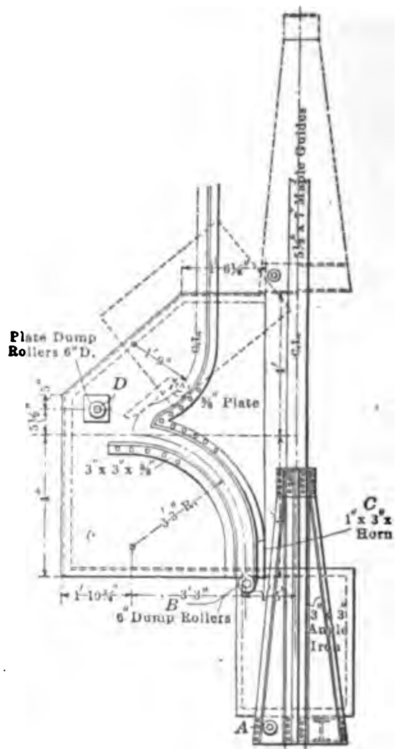

FIG. 37. KIMBERLEY SKIP.

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A skip used in an inclined shaft is shown in Fig. 34. The skip is dumped by having a 4 in tread on the front wheels and a 7 in. tread on the rear wheels. This skip has a capacity of 2 tons. In Fig. 35 the rear wheels have projecting axles which cause the skip to turn nearly upside down.

A skip for a shaft nearly vertical is shown in Fig. 36. The tilting of the skip is positive so that it readily rights itself. The skip for a vertical shaft in Fig. 37 is known as a Kimberley skip, and is the type commonly used for vertical shafts. The skip is turned on axle *A* by

FIG. 38. SELF-DUMPING SKIP.

FIG. 39. WATER SKIP.

means of rollers *B*, taking a course away from the vertical guides as described by the angle iron guides, until the horns *C* intercept rollers *D*, whereupon the skip rollers *B*, are elevated to the upper angle iron guides.

Details of the skip used by the Copper Queen Mining Company are shown in Chapter VI, Fig. 91; and at the Sibley mine in Fig. 106.

A self dumping skip manufactured by the Wellman-Seaver-Morgan Company is shown in Fig. 38, while a water skip is shown in Fig. 39.

OVER WINDING.—Accidents due to over winding cause much property damage, and occasionally great loss of life. Special precau-

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tions should be taken to prevent accidents, and the safety devices should be inspected frequently and kept in repair.

Much interesting and valuable information on hoisting and safety devices is given in the Report of a Royal Commission of the Transvaal on "The Use of Winding Ropes, Safety Catches, and Appliances in Mine Shafts," 1907. The following recommendations are of interest.

1. A factor of safety of 6 is sufficient for hoisting ropes (direct pull). Where men are hoisted, the load should not exceed 85 per cent of the regular load.

2. The two weakest parts in a rope which has been in use for some time are (*a*) near the top of the cage, and (*b*) between the drum and the head sheave when the cage is at the bottom of the shaft.

3. Catch beams in the head gear are approved. In case of over winding the lever arms of these appliances automatically fall inwards after the cage passes upwards and prevents it from descending.

4. Converging guides near the sheave are recommended.

SAFETY HOOKS.—Safety hooks are placed above the skip or cage, and are so designed that in case of over winding the hooks will become detached and the skip or cage will be held by a safety case placed below the sheaves. Many safety detaching hooks have been devised and used with more or less satisfactory results. The safety detaching hook shown in Fig. 40 and in Table IV, is one of the best known and gives excellent results. The device consists of two parts, the hook proper and the case.

Safety. Hook.—The hook consists of three plates pivoted together on a central pin as shown in Fig. 40. The lower ends of the plates are so shaped that the two outer plates project beyond the main portion of the plates on one side, the middle plate projecting in the opposite way. In the lower end of the plates there is an oblong hole of such shape that when the plates are placed in their proper position the three holes form a circle into which is fitted a pin and to which is connected the cage. At the upper end of each of the plates there is a slot, and of such shape, that when all the plates are riveted together it will also form a complete circular hole, into which is fitted a pin and clevis for the rope. The upper ends of the plates are provided with ears, which under running conditions are mutually covered. The plates when

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properly located, are held in place by a soft copper rivet placed just above the center pin.

Case.—The case, Fig. 40, is a circular casting of iron, partly conical and partly cylindrical and is designed to be of ample size to withstand the stresses due to a loaded cage dropping on it from a height of two

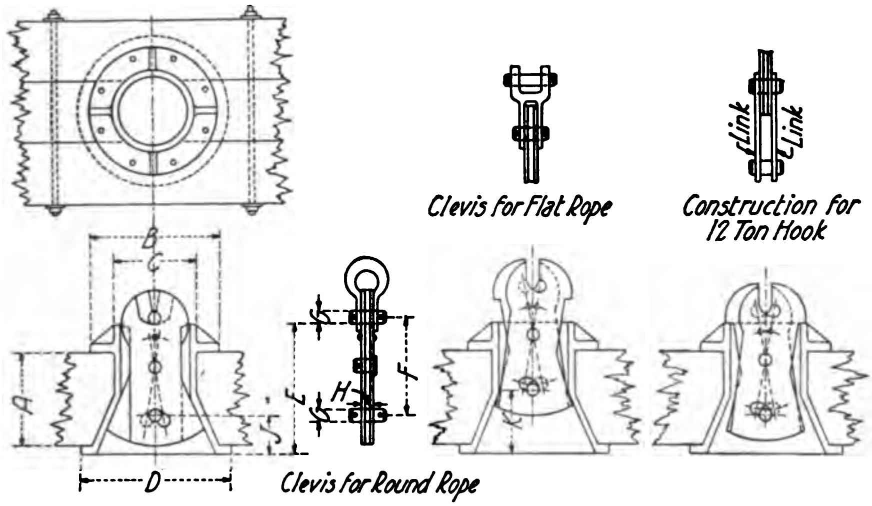

FIG. 40. SAFETY DETACHING HOOKS; WELLMAN-SEAVER-MORGAN COMPANY.

TABLE IV.

DETAILS OF SAFETY DETACHING HOOKS. WELLMAN-SEAVER-MORGAN COMPANY.

Max. Safe Load, Tons.	Dimensions in Inches.										Shipping Weights, Lbs.			
	A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*J*	*K*	Cast Iron.	Forging.	Struct. Steel.	C.R.S.
4	$8\frac{1}{2}$	14	8	18	14	$10\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{3}{4}$	3	7	190	13	39	5
6	12 16	$16\frac{1}{2}$	$10\frac{3}{4}$	$20\frac{1}{2}$	19	14	$1\frac{3}{4}$	$1\frac{7}{8}$	4	11	325	13	96	8
8	18	25	16	29	25	19	$2\frac{1}{16}$	$2\frac{1}{8}$	6	13	875	32	183	28
12	24	$30\frac{5}{8}$	$20\frac{5}{8}$	$36\frac{5}{8}$	33	23	$3\frac{7}{16}$	$2\frac{1}{2}$	9	$17\frac{1}{4}$	1,325	100	351	67

feet. The case is fastened to the head frame as close as possible to the head sheave, leaving sufficient space to allow for the operation of the hook.

The operation of the safety hook is very simple. The cage is hoisted high enough to enter the hook into the case, the flaring sides of

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the hook meeting those of the case are closed just like a pair of scissors. This action shears off the copper rivet which brings the slots in the upper end of the plate in line, allowing the clevis to pull out; the ears on the plate are thrown out far enough so that as the cage drops the ears catch on the upper edge of the case, holding the cage in place. The curved holes near the bottom of the plates are so designed that in the released position the plates are again locked and cannot be changed until the weight of the cage has been taken off of the hook. It is desirable that hooks should be tested at frequent intervals.

Safety hooks were used on the steel head frames for the Union Shaft, Fig. 98, and for the Cia. Minera de Penoles, Fig. 95, constructed by the Wellman-Seaver-Morgan Company. For descriptions of several other safety hooks and safety devices see Ihlseng's "Manual of Mining," also the Report of the Royal Commission of the Transvaal on "The Use of Winding Ropes, Safety Catches, and Appliances in Mine Shafts."

SHEAVES.—Sheaves for head frames are made with cast spokes or with wrought iron spokes, the latter being generally used for large sizes. The tread of the sheaves may be of iron or may be lined with wood, which reduces the wear on the hoisting rope.

The approximate weights of sheaves for round rope are given in Table V and Table VI, while the approximate weights of sheaves for flat ropes are given in Table VII and Table VIII.

TABLE V.

SHEAVES FOR ROUND ROPE. ALLIS-CHALMERS COMPANY.

Diameter, In.	Size Rope. In.	Weight Sheave Only, Lbs.	Weight with Shaft and Boxes, Lbs.
18	$\frac{3}{8}$ to $\frac{1}{2}$	86	120
24	$\frac{1}{2}$ to $\frac{5}{8}$	115	190
30	$\frac{5}{8}$	165	315
36	$\frac{3}{4}$	250	430
42	$\frac{3}{4}$	440	665
48	$\frac{3}{4}$ to $\frac{7}{8}$	460	750
60	$\frac{7}{8}$ to 1	900	1,200
66	1	1,100	1,400
72	1 to $1\frac{1}{8}$	1,200	1,800
84	$1\frac{1}{8}$	1,530	2,400
96	$1\frac{1}{8}$ to $1\frac{1}{4}$	1,950	3,030

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TABLE VI.

HEAVY DUTY SHEAVES FOR ROUND ROPE. WROUGHT ARMS.
WELLMAN-SEAVER-MORGAN COMPANY.

Diameter, Ft.	Style of Tread.	Maximum Diameter of Rope, In.	Journals.		Weight.	
			Diameter, In.	Length, In.	Sheave Only, Lbs.	Sheave, Shaft, and Two Flat Boxes, Lbs.
2	Cast	1	$2\frac{7}{16}$	7	250	370
3	Cast	1	$3\frac{1}{2}$	8	350	607
4	Cast	1	4	8	600	961
5	Wrought	1	$4\frac{1}{2}$	8	1,050	1,467
6	Wrought	1	$4\frac{1}{2}$	10	1,500	1,973
7	Wrought	$1\frac{1}{8}$	$5\frac{1}{2}$	10	2,000	2,764
8	Wrought	$1\frac{1}{4}$	6	12	2,700	,914
10	Wrought	$1\frac{3}{8}$	7	13	3,500	5,160
12	Wrought	$1\frac{1}{2}$	8	13	6,100	8,070
14	Wrought	$1\frac{3}{4}$	9	16	8,600	11,580

TABLE VII.

SHEAVES FOR FLAT ROPE. ALLIS-CHALMERS COMPANY.

Diameter, In.	Size Rope, In.	Weight, Sheave Only, Lbs.	Weight with Shaft and Boxes, Lbs.
36	$3 \times \frac{3}{8}$	450	600
48	$3\frac{1}{2} \times \frac{3}{8}$	750	1,000
60	$4 \times \frac{3}{8}$	1,300	1,800
72	$4\frac{1}{2} \times \frac{3}{8}$	1,925	2,800
84	$5 \times \frac{3}{8}$ or $\frac{1}{2}$	2,200	3,260
96	$5\frac{1}{2} \times \frac{1}{2}$	2,400	3,400

TABLE VIII.

SHEAVES FOR FLAT ROPE. WROUGHT ARMS. WELLMAN-SEAVER-MORGAN COMPANY.

Diameter of Sheave, In.	Width of Rope, In.	Journals.		Weight.		Style of Box.
		Diameter, In.	Length, In.	Sheave Only, Lbs.	Sheave, Shaft and Two Boxes, Lbs.	
60	$3\frac{1}{2}$	$3\frac{1}{2}$	8	862	1,200	Post
60	4	$3\frac{1}{2}$	8	890	1,165	Flat
72	$3\frac{1}{2}$	$4\frac{1}{2}$	10	1,520	2,022	Flat
72	4	$4\frac{1}{2}$	10	1,560	2,062	Flat
72	5	$4\frac{1}{2}$	10	1,600	2,102	Flat
72	$5\frac{1}{2}$	$4\frac{1}{2}$	10	1,625	2,127	Flat
84	5	$4\frac{1}{2}$	10	2,000	2,502	Flat
84	6	$4\frac{1}{2}$	10	2,100	2,602	Flat
96	5	$5\frac{1}{2}$	10	2,400	3,429	Post
96	6	$5\frac{1}{2}$	10	2,500	3,529	Post
120	7	7	12	5,400	7,746	Post
120	8	7	12	5,500	7,746	Post

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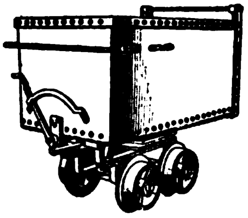
(a)


(b)

FIG. 41.

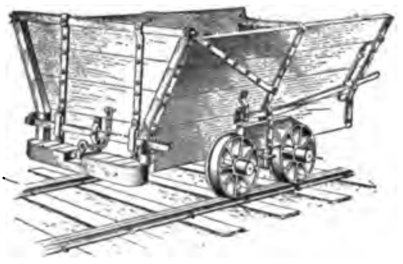

FIG. 42.

FIG. 43.

CARS.—The ore or coal is hauled to the bottom of the shaft in mine cars. Examples of steel cars for use in metal mines are given in Fig. 41, while a car for use in coal mines is shown in Fig. 42 and a steel car in Fig. 43. Details of the coal car used in the coal mines of the Alberta Railway and Irrigation Company are shown in Fig. 136, Chapter VII.

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CHAPTER III.

Stresses in Simple Head Frames.

Introduction.—The stresses in a head frame will depend (1) upon the location of the hoisting engine, (2) upon the height of the head frame, (3) upon the depth of the mine, (4) upon the weight and capacity of the skip, (5) upon the wind pressure, and (6) upon the

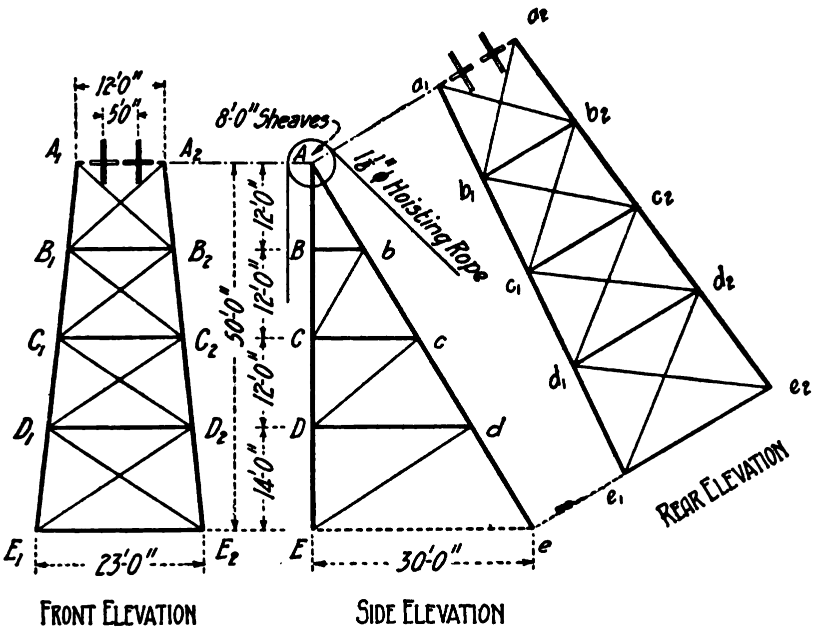

Fig. 44. Steel Head Frame.

design of the head frame itself. The forces acting upon the head frame are (*a*) the pull of the rope under different conditions, (*b*) the weight of the structure, (*c*) the pressure of the wind acting horizontally in any direction.

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The maximum stress due to the hoisting rope will occur when the cage or skip sticks in the shaft, and is equal to the maximum breaking strength of the rope. The wind load may be taken at 30 lbs. per sq. ft. of exposed surface, acting horizontally. Where head frames are placed in positions with an unusual exposure it may be necessary to take the

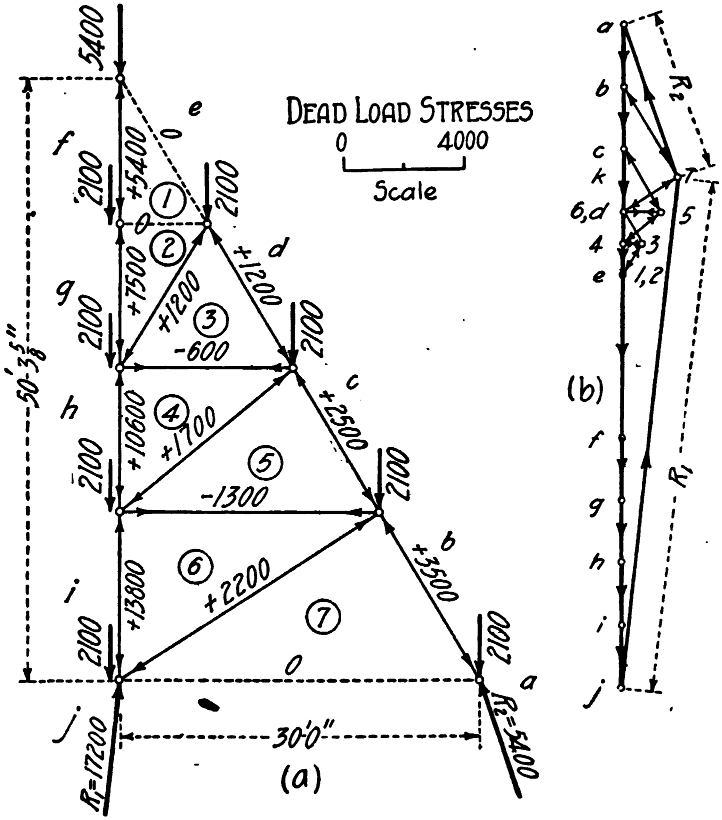

FIG. 45.

wind pressure at 40 lbs. per sq. ft. The weight of the structure should be estimated from the known weight of some similar structure. The weight of the sheaves may be obtained from Tables V to VIII. The resultant of the stresses in the hoisting rope will approximately bisect the angle between the rope and the load for working conditions, and

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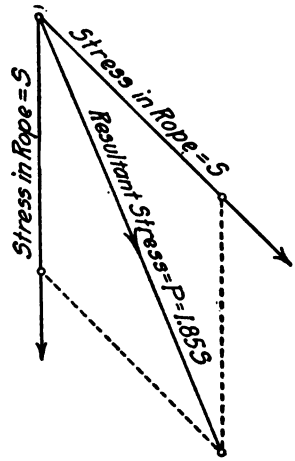
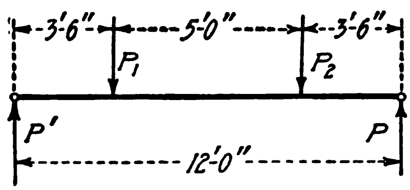
will exactly bisect the angle for maximum conditions. The stresses in head frames of the A-type are usually statically determinate, while the stresses in head frames of the 4-post type are statically indeterminate.

Loading	S	P
Empty Skip	4200	7800
Loaded Skip	13300	24600
Breaking Rope	84000	155200





Case	P_1	P_2	P'	P	E'	E
1	24600	24600	24600	24600	24700	24700
2	7800	155200	50600	112400	50800	113000
3	155200	155200	155200	155200	156000	156000

$$E = P \sec\theta = 1.005\,P,$$
$$E' = P' \sec\theta = 1.005\,P'$$

FIG. 46.

Statically determinate head frames, only, will be considered in this chapter, while statically indeterminate head frames will be considered in Chapter V. For the calculations of wind stresses in a trestle bent, see Fig. 154, Chapter VIII.

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STRESSES IN AN A-TYPE STEEL HEAD FRAME.—Given a mine 1,500 ft. deep, weight of skip 1,000 lbs., weight of load 4,000

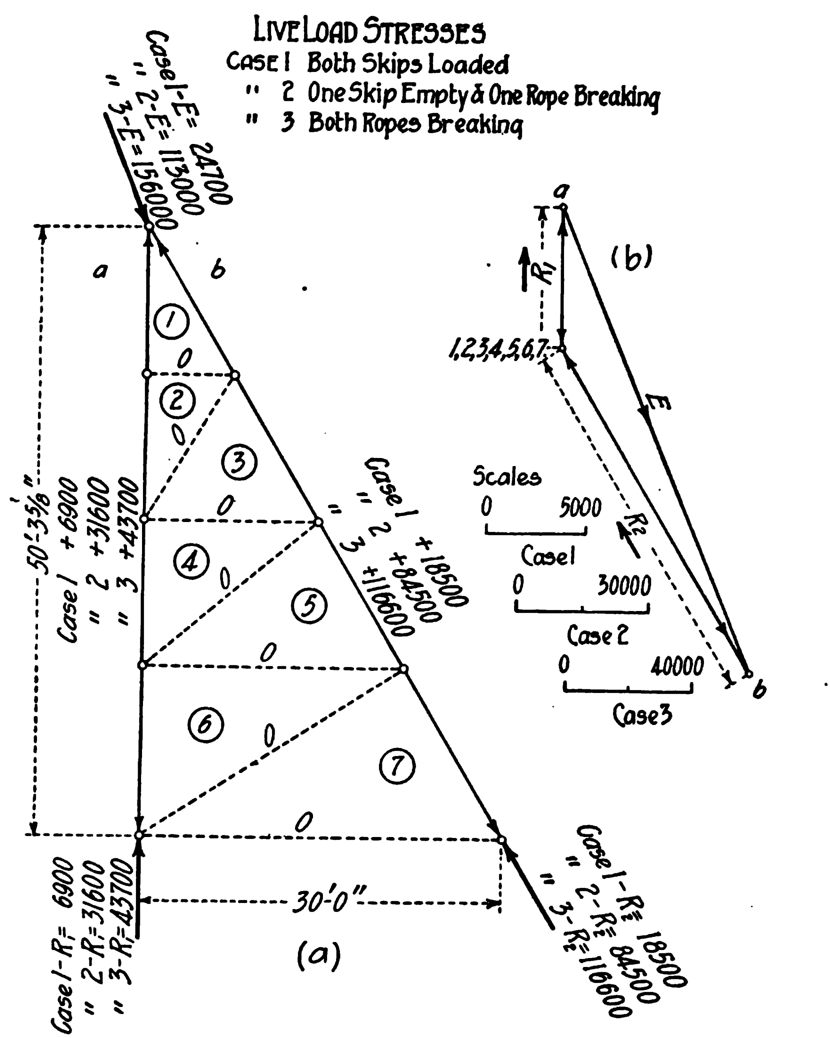

FIG. 47.

lbs., weight of $1\frac{1}{8}$-in. round rope 2 lbs. per ft. The working load in the hoisting rope from (1) will be

$$K = 2W + R + (W + R)f \tag{1}$$
$$= 10{,}000 + 3{,}000 + 8{,}000 \times 0.02$$
$$= 13{,}160 \text{ lbs.}$$

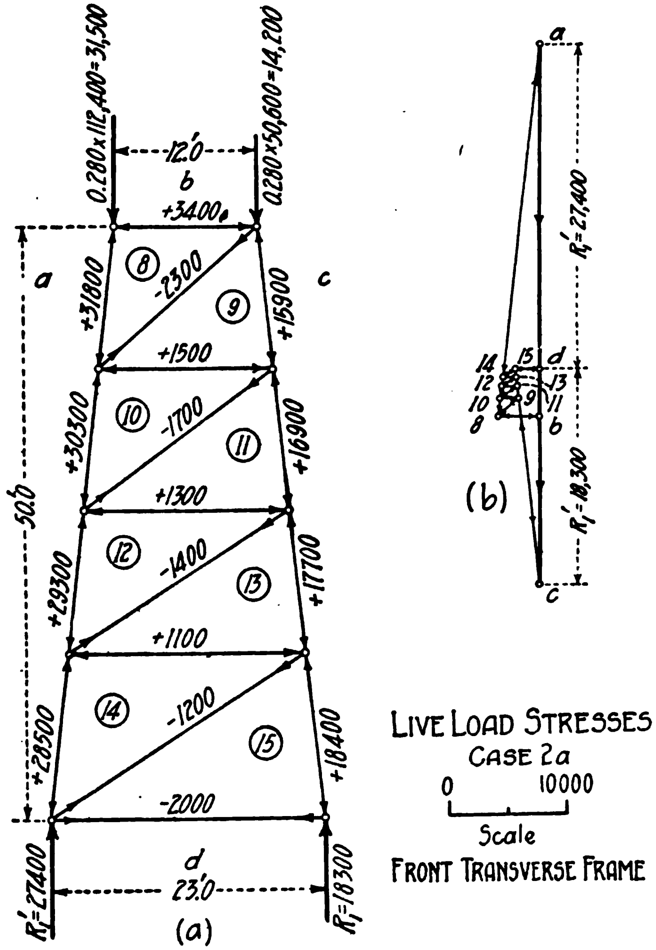

FIG. 48.

From Fig. 26 with 8-ft. sheaves the working load of a $1\frac{1}{8}$-in. rope is 14,000 lbs. The breaking load of a $1\frac{1}{8}$-in. rope from Table I is 84,000 lbs. The weight of the steel head frame will be assumed as 42,000 lbs. The weight of the sheaves and blocks will, from Table V,

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be equal to 2,400 lbs. The wind load will be taken at 30 lbs. per sq. ft., acting horizontally.

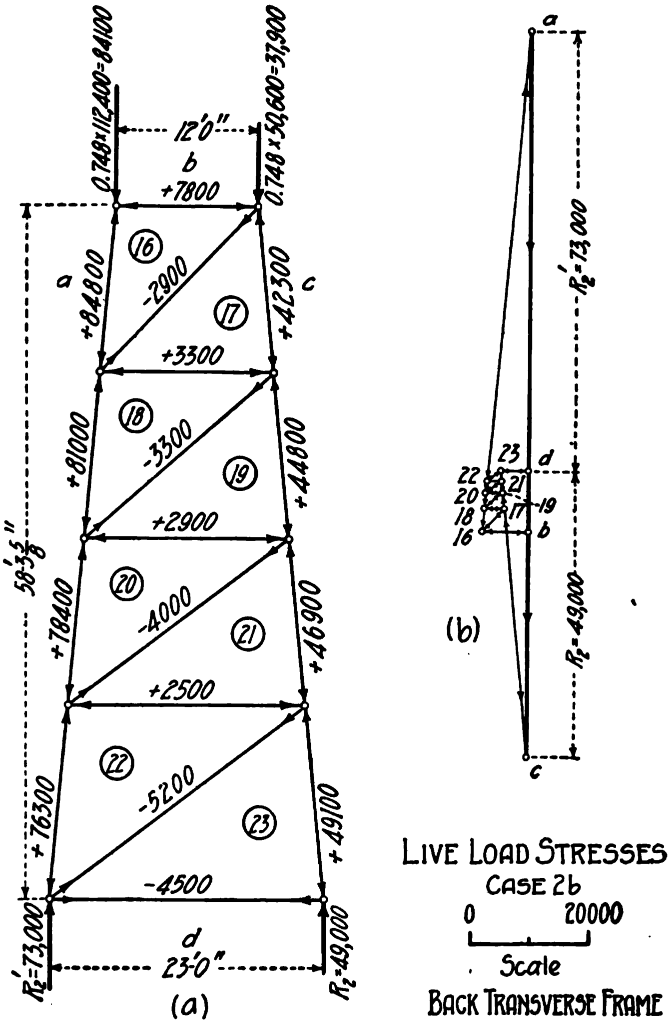

Fig. 49.

The dimensions of the head frame will be assumed as in Fig. 44. The hoisting drum will be placed so that the upper rope will make an angle of 45 degrees with the horizontal.

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CALCULATION OF STRESSES.—The stresses will be calculated for the following conditions: (1) dead load; (2) live load, and (3) wind load.

Dead Load Stresses.—The dead loads will be assumed as concentrated at the joints, and it will be assumed that the side A-frames carry

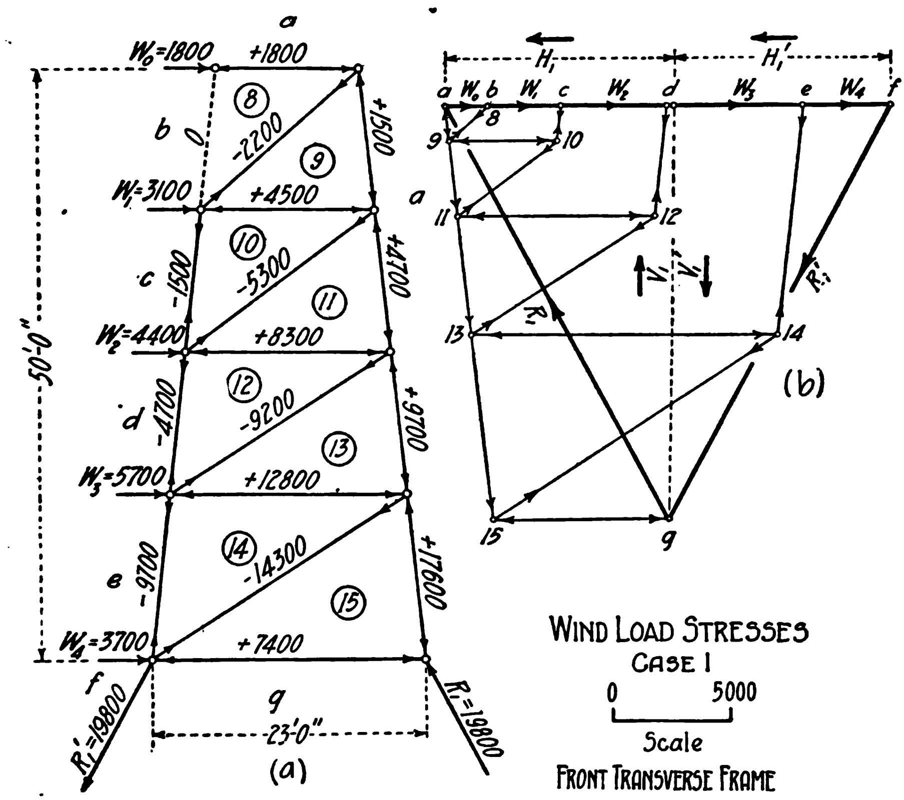

FIG. 50.

the dead loads. The loads will be taken parallel to the plane of the frames, giving a slant height for the frame of 50 ft. 3⅝ in. The stresses have been calculated by graphic resolution in (*b*) and are as shown on the frame in (*a*), Fig. 45. The loads in the force polygon in (*b*), Fig. 45, are laid off from point *e*, the loads on the left being laid off downwards, *e–f* = 5,400 lbs., *f–g* = 2,100 lbs., *g–h* = 2,100 lbs., *h–i* = 2,100 lbs.,

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and *i–j* = 2,100 lbs.; also *e–d* = 2,100 lbs., *d–c* = 2,100 lbs., *c–b* = 2,100 lbs., and *b–a* = 2,100 lbs. Then stress in *1–f* = 5,400 lbs., stress in *f–g* = 5,400 + 2,100 = 7,500 lbs. Load *e–d* is held in equilibrium by stresses in *d–3* and *2–3*. Closing the force polygon *e–3–d* in (*b*) gives

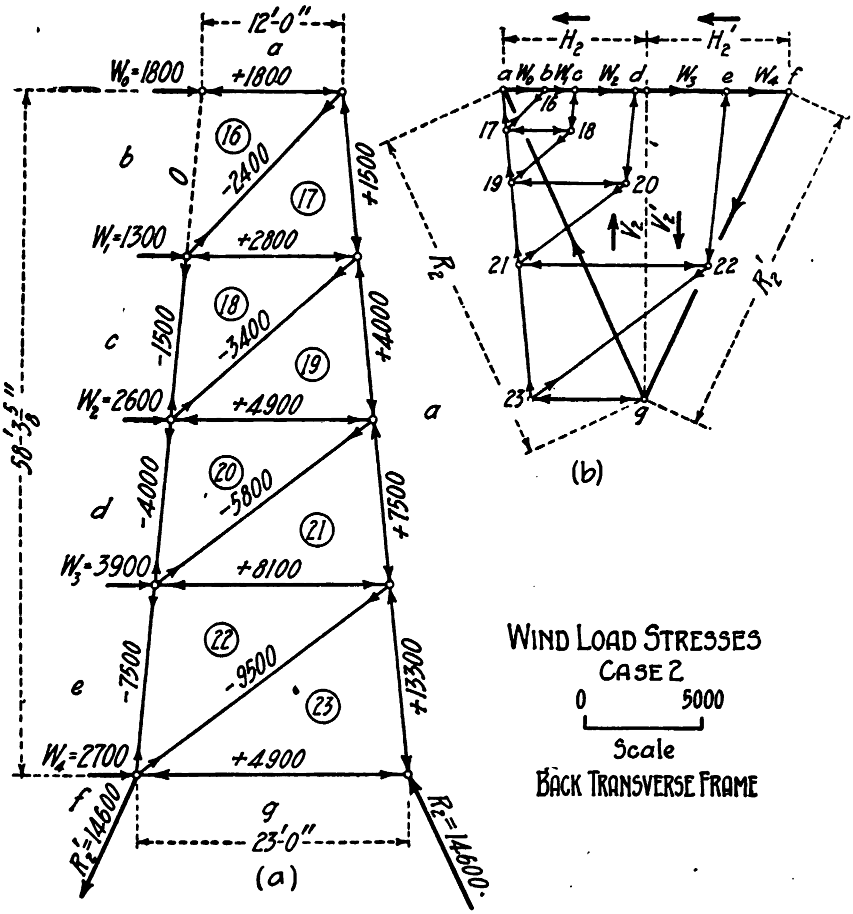

FIG. 51.

stress *d–3* = + 1,200 lbs., and stress *2–3* = + 1,200 lbs. The stresses in *3–4* and *4–h* are calculated by drawing force polygon *2–3–4–h–g–2* in stress diagram (*b*). The remainder of the stresses are calculated in a similar manner.

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Live Load Stresses.—The stresses will be calculated for the following conditions: Case 1, both skips loaded; case 2, one skip loaded, one skip empty, and one rope breaking; case 3, both ropes breaking.

The resultant stresses, P, for different loads, S, are given in Fig. 46. The loads P_1 and P_2 on the sheaves, Fig. 46, for case 1 are 24,600 lbs.,

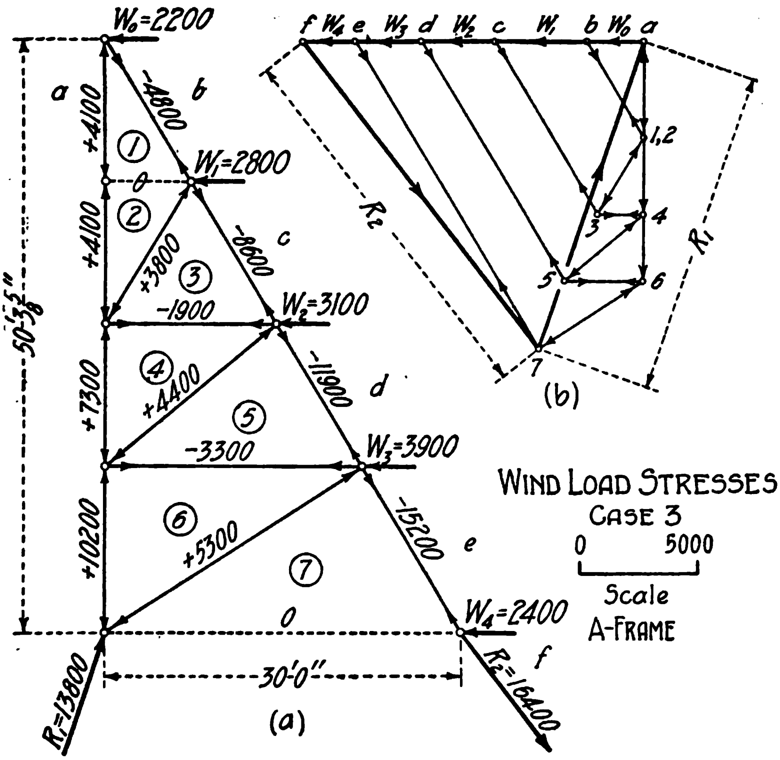

FIG. 52.

the reactions at the ends of the sheave girder are $P' = P = 24{,}600$ lbs.; while the inclined components E' and E are each 24,700 lbs. For case 2, $P_1 = 7{,}800$ lbs., $P_2 = 155{,}200$ lbs., $P' = 50{,}600$ lbs., $P = 112{,}400$ lbs., $E' = 50{,}800$ lbs., and $E = 113{,}000$ lbs. For case 3, $P_1 = P_2 = P' = P = 155{,}200$ lbs., $E' = E = 156{,}000$ lbs.

Live Load Stresses in the Posts.—The live load stresses in the posts for cases 1, 2 and 3 are calculated in Fig. 47 by graphic resolution.

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The A-bent has been developed, and it is assumed that none of the loads are transferred by the transverse frames. For case 2, the transverse frames transfer part of the loads to the opposite A-frame so that the stresses in the posts near the bottom are less than given in Fig. 47. There are no stresses in the bracing due to live loads.

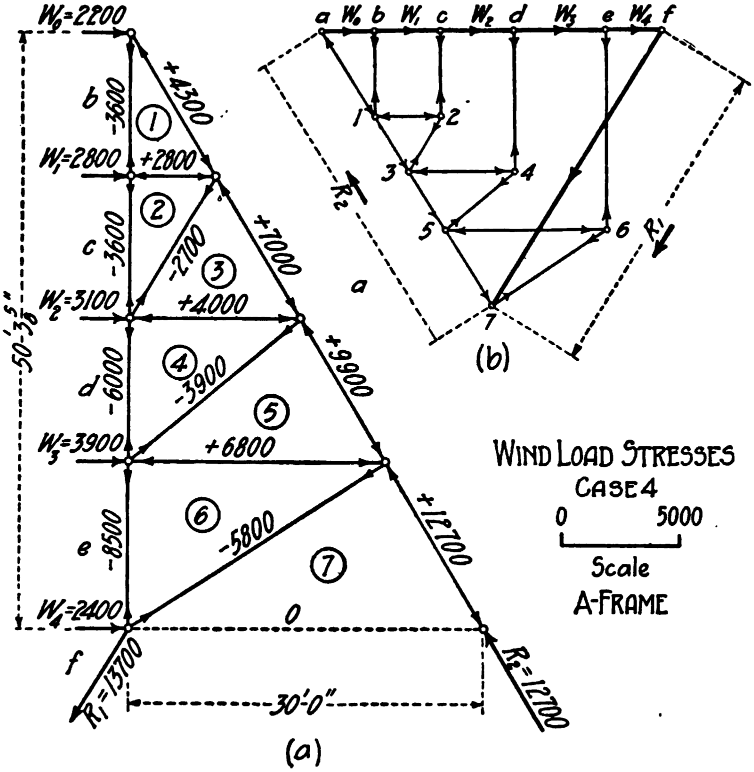

FIG. 53.

Live Load Stresses in the Transverse Frames.—The maximum stresses in the bracing of the front and back transverse frames will occur for case 2. The loads on the front transverse frame are 31,500 lbs. and 14,200 lbs. The stresses are calculated by graphic resolution in (*b*) and the stresses are written on the frames in (*a*), Fig. 48. The stresses in the back transverse brace are calculated in a similar manner in Fig. 49.

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Wind Load Stresses.—The wind pressure was assumed as 30 lbs. per sq. ft. acting horizontally, and it is assumed that the head frame is closed in so that the full wind pressure will come on the exposed surface. The wind load stresses have been calculated by graphic resolution, and are given in Fig. 50, Fig. 51, Fig. 52, and Fig. 53.

For the design of the sections in this head frame, see Chapter VI.

TABLE IX.

STRESSES IN A-TYPE HEAD FRAME.

Members.	Dead Load Stresses.	Live Load Stresses.			Wind Load Stresses.			
		Case 1.	Case 2.	Case 3.	Case 1.	Case 2.	Case 3.	Case 4.
Front posts	+13,800	+ 6,900	+31,600	+ 43,700	+17,600 − 9,700		+10,200	− 8,500
Back braces	+ 3,500	+18,500	+84,500	+116,600		+13,300 − 7,500	−15,200	+12,700
Bracing *6–7*	+ 2,200						+ 5,300	− 5,800
4–5	+ 1,700						+ 4,400	− 3,900
2–3	+ 1,200						+ 3,800	− 2,700
Strut *5–6*	− 1,300						− 3,300	+ 6,800
3–4	− 600						− 1,900	+ 4,000
1–2	0						0	+ 2,800
Bracing *8–9*			− 2,300		− 2,200			
10–11			− 1,700		− 5,300			
12–13			− 1,400		− 9,200			
14–15			− 1,200		−14,300			
Strut *9–10*			+ 1,500		+ 4,500			
11–12			+ 1,300		+ 8,300			
13–14			+ 1,100		+12,800			
15–g			− 2,000		+ 7,400			
Bracing *16–17*			− 2,900		− 2,400			
18–19			− 3,300		− 3,400			
20–21			− 4,000		− 5,500			
22–23			− 5,200		− 9,500			
Strut *17–18*			+ 3,300		+ 2,800			
19–20			+ 2,900		+ 4,900			
21–22			+ 2,500		+ 8,100			
23–d			− 4,500		+ 4,900			

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CHAPTER IV.

Stresses in Statically Indeterminate Structures.

Introduction.—In all structures the external forces acting on the structure must be in equilibrium, and the internal stresses must be in equilibrium. The external forces acting on a structure are the loads which the structure carries and the reactions which equilibrate the loads. In statically determinate structures the reactions may be calculated by applying the fundamental principles of static equilibrium to the external forces and the reactions, (*a*) the sum of the horizontal components of forces equal zero, (*b*) the sum of the vertical components of forces equal zero, and (*c*) the moments of the forces about any point in the plane of the forces must be equal to zero. It is usually only necessary to apply the condition (*c*) for moments of forces in calculating the reactions of simple structures carrying vertical loads. Having calculated the external forces the internal stresses in statically determinate structures may be calculated either by algebra or by graphics, by applying conditions for equilibrium (*a*) and (*b*) (resolution), or condition for equilibrium (*c*) (moments). (For the calculation of statically determinate structures, see Chapter VIII, and the author's "The Design of Steel Mill Buildings.")

Structures may be statically indeterminate externally, or internally, or both externally and internally. A structure is statically indeterminate externally when the reactions depend upon the rigidity and design of the structure, as for example, a beam supported upon more than two supports, or a truss of a swing bridge which is supported on three or four supports. A structure is statically indeterminate internally when it is not possible to reduce the moment equations for all members so that each equation will contain one unknown force; or reduce the resolution equations so that condition equations (*a*) and (*b*) will each contain not more than two unknown stresses, each equation containing

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the same unknowns. The stresses in a statically indeterminate structure depend upon the rigidity of the structure, and it is therefore necessary to know the exact sizes and dimensions of the members before the stresses can be calculated.

If a structure is acted upon by an external force which produces stresses in the structure, the structure will deflect under the load, and the stresses in the members will cause the compression members to shorten and the tension members to lengthen. The external work will be equal to one half the product of the load and the deflection of the structure under the load—it is assumed that the external load has been gradually applied, if the load were suddenly applied the work would be equal to the product of the force and the deflection, which would double the stresses due to a load gradually applied and would cause the structure to vibrate until it came to rest under the conditions first assumed, providing the stresses in the structure have not exceeded the elastic limit of the material. The internal work in the structure will be equal to the sum of one half the stress in each member multiplied by the deformation of the member due to the stress. Now, since the structure simply stores up the energy due to the application of the external load, the work of the external load must be equal to the work of the internal stresses. A fourth condition for equilibrium of a structure then is, (*d*) that the work of the external forces is equal to the work of the internal stresses.

Since the work of the external forces is always equal to the work of the internal stresses, it follows that the stresses in any structure due to any external load or loads will be such as to make the work of distortion a minimum, and be equal to the work of the external forces. This principle is called the "Theory of Least Work."

STRESSES IN A FRAMEWORK WITH ONE REDUNDANT MEMBER. First Method.—The framework in (*a*), Fig. 55, is a two-hinged arch with a horizontal tie, one end being fixed and the other end resting on frictionless rollers. The stress in the tie *3–c* will depend upon the rigidity of the triangular truss and of the tie, the deformation of the tie *3–c* being equal to the horizontal deformation of the

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arch. The arch carries a load W, which produces vertical reactions R_1 and R_2, which are statically determinate.

In (*b*), Fig. 55, assume that the tie *3–c* is replaced by the horizontal stress, S_1, acting in place of the tie. Assume that all members of the framework are rigid except member *1–3*, which is increased in length, δ, under the action of the load, W, and that the right end of the arch moves a distance Δ to the right Let s be the horizontal force neces-

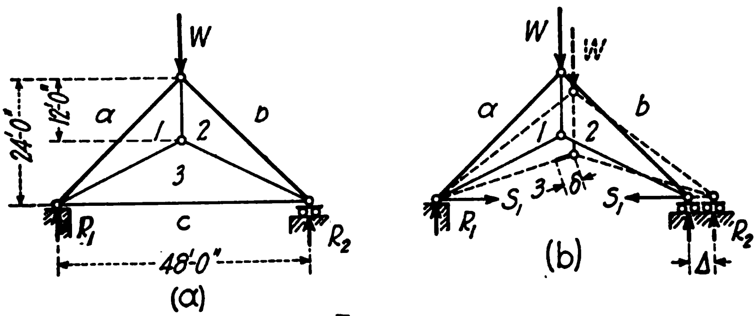

FIG. 55.

sary to bring the right reaction back to its original position. Now, the internal work in the member *1–3* will be $\frac{1}{2}s \cdot U \cdot \delta$, where U is the stress in *1–3* due to a stress $S_1 =$ unity, and the external work in bringing the right abutment back to its original position is $\frac{1}{2}s \cdot \Delta$. Now, since the internal work is equal to the external work

$$\Delta = U \cdot \delta \tag{9}$$

But $\delta = \dfrac{S_{1-3} \cdot L}{A \cdot E}$, where $L =$ length of the member in inches, $A =$ area of a cross-section of the member in square inches, $E =$ modulus of elasticity of member in pounds per square inch, and $S_{1-3} =$ stress in member *1–3* due to load W, and substituting in (9)

$$\Delta = \frac{S_{1-3} \cdot U \cdot L}{A \cdot E} \tag{10}$$

Now, if all the members in turn are assumed to carry stress, the deformation of the right end of the truss will be

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$$\Delta_1 = \Sigma \frac{S \cdot U \cdot L}{A \cdot E} \tag{11}$$

Now, if $s \cdot U$ is the stress in member *1–3* due to the stress s acting in line with S_1 at the right hinge, then in the same manner as above

$$\Delta = U \cdot \delta \tag{9}$$

and since

$$\delta = \frac{s \cdot U \cdot L}{A \cdot E}$$

$$\Delta = \frac{s \cdot U^2 \cdot L}{A \cdot E} \tag{12}$$

Now, if all the members in turn are assumed to carry stress, the right end of the truss will be brought back a distance

$$\Delta' = \Sigma \frac{s \cdot U^2 \cdot L}{A \cdot E} \tag{13}$$

Now $\Sigma s = S_1$, and

$$\Delta' = S_1 \Sigma \frac{U^2 \cdot L}{A \cdot E} \tag{14}$$

Now the deformation of the tie 3–c is $\Delta'' = (S_1 \cdot l)/(A \cdot E)$, where l = length of 3–c. Then since the deformation of the tie = deformation of right end of truss, $\Delta_1 = \Delta' + \Delta''$, and

$$S_1 = \frac{\Sigma \frac{S \cdot U \cdot L}{A \cdot E}}{\Sigma \frac{U^2 \cdot L}{A \cdot E} + \frac{l}{A \cdot E}} \tag{15}$$

Problem 1.—To explain the application of the above method the stresses will be calculated in the two-hinged arch given in (a), Fig. 56, where W = 20,000 lbs., and the areas of the members are as given in Table X, column 2.

Solution.—The details of the solution are as follows: (1) Assume that member *3–c* is not acting, and calculate the stresses, S', in the truss in (a), Fig. 56, assuming that it is free to move at the right reaction

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and carries a load of $W = 20,000$ lbs. The stresses are calculated by graphic resolution in (*b*), Fig. 56, and the stresses are given in Table X, column 3. The lengths, L, of the members are given in column 4, Table X; the deformation of the members, for $E = 30,000,000$, are

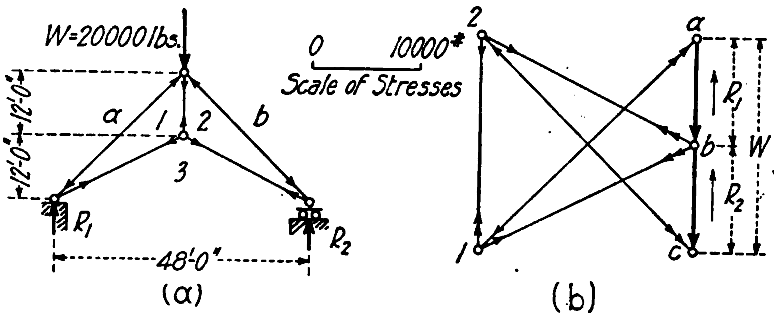

FIG. 56.

given in column 5; the values of U—calculated as will be explained presently—are given in column 6; and the values of $(S' \cdot U \cdot L)/(A \cdot E)$ are given in column 7. The total horizontal deformation of the truss under the load W, is 1.142 in.

(2) The stresses in the framework due to stress $S_1 =$ unity, are then calculated. It was not convenient to use a unit load and a stress

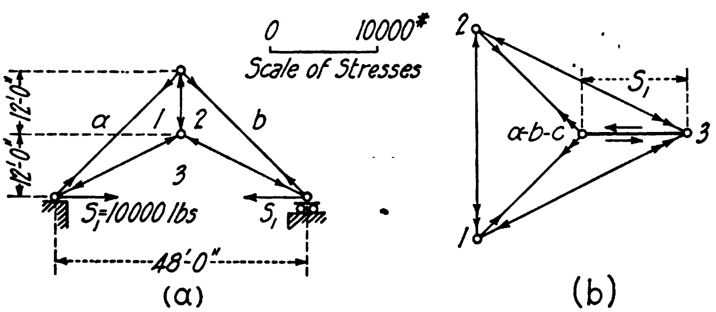

FIG. 57.

of $S_1 = 10,000$ lbs. was taken, and the stresses U in each member were then equal to the calculated stresses divided by 10,000. The stresses in the members due to $S_1 = 10,000$ lbs. are called S''. The stresses S'' for each member are calculated in (*b*), Fig. 57, by graphic resolution, and

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TABLE X.

DEFORMATION OF SIMPLE TRUSS FOR $W = 20,000$ LBS.

1	2	3	4	5	6	7
Member.	Area, A, Sq. In.	Stress, S', Lbs.	Length, L, In.	$\frac{S' \cdot L}{A \cdot E}$	U	$\frac{S' \cdot U \cdot L}{A \cdot E}$
a–1	2.00	+ 28,000	400	+ 0.187	— 1.42	— 0.266
1–3	2.00	— 22,000	320	— 0.117	+ 2.20	— 0.257
1–2	2.00	— 20,000	144	— 0.048	+ 2.00	— 0.096
2–b	2.00	+ 28,000	400	+ 0.187	— 1.42	— 0.266
2–3	2.00	— 22,000	320	— 0.117	+ 2.20	— 0.257

$$\Sigma \frac{S' \cdot U \cdot L}{A \cdot E} = -1.142 \text{ in.}$$

TABLE XI.

DEFORMATION OF SIMPLE TRUSS FOR $S_1 = 10,000$ LBS.

1	2	3	4	5	6	7
Member.	Area, A, Sq. In.	Stress, S'', Lbs.	Length, L, In.	$\frac{S'' \cdot L}{A \cdot E}$	U	$\frac{S'' \cdot U \cdot L}{A \cdot E}$
a–1	2.00	—14,200	400	—0.095	—1.42	0.135
1–3	2.00	+22,000	320	+0.117	+2.20	0.257
1–2	2.00	+20,000	144	+0.048	+2.00	0.096
2–b	2.00	—14,200	400	—0.095	—1.42	0.135
2–3	2.00	+22,000	320	+0.117	+2.20	0.257
					$\Sigma \frac{S'' \cdot U \cdot L}{A \cdot E}$	$=0.880$ in
3–c	0.50	—10,000	576	—0.190	—1.00	0.384

$$S_1 \left(\frac{0.384}{10,000} + \frac{0.880}{10,000} \right) = -1.142. \qquad S_1 = -9,060 \text{ lbs.}$$

are given in column 3, Table XI. The stress (U) in each member due to a stress of unity in the horizontal tie are equal to stress S'' divided by 10,000, and are given in column 6. The deformation of the framework for $S_1 = 10,000$ lbs. is 0.880 in. The deformation of the tie 3–c for a stress of 10,000 lbs. will be 0.384 in.

(3) Now the deformation of the framework will be equal to the deformation of the tie. The total deformation will be equal to deformation of the tie for a stress of one lb., plus the deformation of the framework for a stress of one lb., both multiplied by the true stress in the tie, and

$$S_1 \left(\frac{0.384}{10,000} + \frac{0.880}{10,000} \right) = -1.142 \text{ in.,} \quad \text{and} \quad S_1 = -9,060 \text{ lbs.} \quad (16)$$

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The stresses in the two-hinged arch with a stress of $S_1 = -9{,}060$ lbs. in the tie, have been calculated in (*b*), Fig. 58, by graphic resolution. The stresses in the members of the two-hinged arch may be cal-

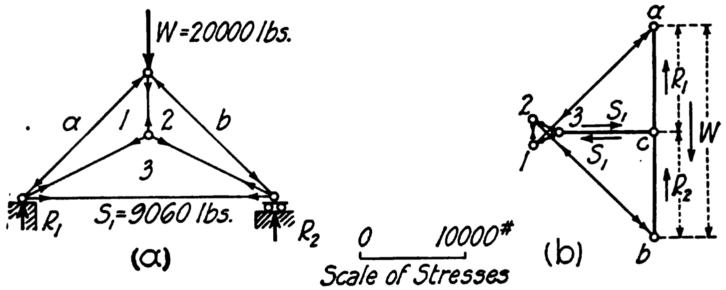

FIG. 58.

culated algebraically, by adding algebraically the stress S' in Table X to the stress $U \times 9{,}060$ for the corresponding member in Table X. For example the stress in member *a–1* in the two-hinged arch $= +28{,}000 - 1.42 \times 9{,}060 = +15{,}150$ lbs.

TABLE XII.

DEFORMATION OF FRAMEWORK.

1	2	3	4	5	6	7
Member.	Area, A, Sq. In.	Stress, S, Lbs.	Length, L, In.	$\frac{S \cdot L}{A \cdot E}$	U	$\frac{S \cdot U \cdot L}{A \cdot E}$
a–1	2.00	+15,150	400	0.101	−1.42	−0.144
1–3	2.00	− 2,100	320	−0.011	+2.20	−0.026
1–2	2.00	− 1,900	144	−0.004	+2.00	−0.008
2–b	2.00	−15,150	400	0.101	−1.42	−0.144
2–3	2.00	− 2,100	320	−0.011	+2.20	−0.026
						$\Sigma \frac{S \cdot U \cdot L}{A \cdot E} = -0.348$
Deformation of tie *1–3*.						
3–c	0.50	−9,060	576	−0.34	−1.00	+0.348

The deformation of the framework has been calculated in Table XII. It will be seen that the deformation of the framework is equal to the deformation of the tie, which checks the solution.

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The deformations of the structure may also be calculated by graphics—see Chapter XIV, "The Design of Steel Mill Buildings."

Second Method.—The second method depends upon the theorem due to Castigliano, commonly called the "Theory of Least Work," which will now be explained.

THEORY OF LEAST WORK.—In Fig. 59 a beam or truss is loaded with two loads, P_1 and P_2, which produce deflections y_1 and y_2.

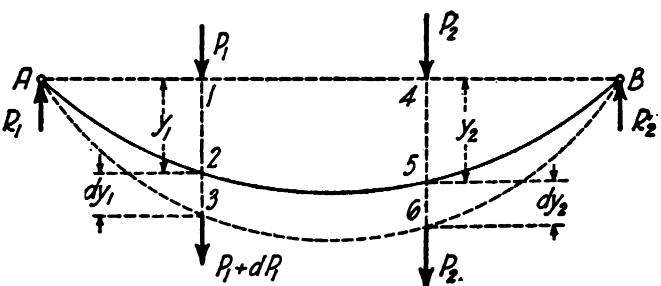

FIG. 59.

The external work, W, done by the two loads, if both P_1 and P_2 are increased gradually from zero to full load, will be equal to the internal work, and will be

$$W = \tfrac{1}{2}P_1 \cdot y_1 + \tfrac{1}{2}P_2 \cdot y_2 \qquad (17)$$

Now let P_1 be increased by a differential load dP_1, producing a further deflection of dy_1 under P_1, and dy_2 under P_2. The external work will then be

$$W + dW = \tfrac{1}{2}P_1 \cdot y_1 + P_1 \cdot dy_1 + \tfrac{1}{2}dP_1 \cdot dy_1 + \tfrac{1}{2}P_2 \cdot y_2 + P_2 \cdot dy_2 \qquad (18)$$

since P_1 and P_2 act through the entire distances dy_1 and dy_2, respectively.

Subtracting (17) from (18), and dropping the product of differentials

$$dW = P_1 \cdot dy_1 + P_2 \cdot dy_2 \qquad (19)$$

Now if $P_1 + dP_1$ and P_2 are increased gradually from zero to full values,

$$W + dW = \tfrac{1}{2}(P_1 + dP_1)(y_1 + dy_1) + \tfrac{1}{2}P_2(y_2 + dy_2) = \tfrac{1}{2}P_1 \cdot y_1$$
$$+ \tfrac{1}{2}P_1 \cdot dy_1 + \tfrac{1}{2}dP_1 \cdot y_1 + \tfrac{1}{2}dP_1 \cdot dy_1 + \tfrac{1}{2}P_2 \cdot y_2 + \tfrac{1}{2}P_2 \cdot dy_2 \qquad (20)$$

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Subtracting (17) from (20), and dropping products of differentials,

$$dW = \tfrac{1}{2}P_1 \cdot dy_1 + \tfrac{1}{2}dP_1 \cdot y_1 + \tfrac{1}{2}P_2 \cdot dy_2 \tag{21}$$

Now multiply (21) by 2, and subtract from (19), and

$$dW = dP_1 \cdot y_1,$$

and

$$\frac{dW}{dP_1} = y_1 \tag{22}$$

which is Castigliano's first theorem.

Castigliano's first theorem then is, "*The derivative of the total internal work, W, with respect to any one load, or external force, P_1, is equal to the displacement, y_1, in the direction of this load, P_1.*"

Castigliano's second theorem is "*The derivative of the work, W, due to any load, P_1, with respect to the displacement, y_1, in the direction of P_1, is equal to the load, P_1.*"

This can be proved from equation (17) by making P_2 equal zero, when

$$\frac{dW}{dy_1} = P_1 \tag{22'}$$

The second theorem of Castigliano is of little use, the first theorem being the more important one.

Application to a Framed Structure.—Now let S_1, S_2, S_3, etc.; A_1, A_2, A_3, etc., and L_1, L_2, L_3, etc., be the stresses, sectional areas, and the lengths of the members of a framed structure when carrying a load; and let

$$\frac{L_1}{A_1 \cdot E} = B_1;\ \frac{L_2}{A_2 \cdot E} = B_2;\ \frac{L_3}{A_3 \cdot E} = B_3, \text{ etc.}$$

Then the work of deformation will be

$$W = \tfrac{1}{2}S_1^2 \cdot B_1 + \tfrac{1}{2}S_2^2 \cdot B_2 + \tfrac{1}{2}S_3^2 \cdot B_3 + \text{etc.} \tag{23}$$

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Differentiating (23) with reference to S_1, and

$$\frac{dW}{dS_1} = B_1 \cdot S_1\left(\frac{dS_1}{dS_1}\right) + B_2 \cdot S_2\left(\frac{dS_2}{dS_1}\right) + B_3 \cdot S_3\left(\frac{dS_3}{dS_1}\right) + \text{etc.} \quad (24)$$

$$= \sum B \cdot S\left(\frac{dS}{dS_1}\right) = y_1 \quad (25)$$

STRESSES IN A FRAMEWORK WITH ONE REDUNDANT MEMBER. Second Solution.—The framework in (*a*), Fig. 60, carries a load, $P = 20{,}000$ lbs. The area of each member is $A = 2$ sq. in. The framework would be stable with either member AD (S_5) or member CB (S_4), and has therefore one redundant member.

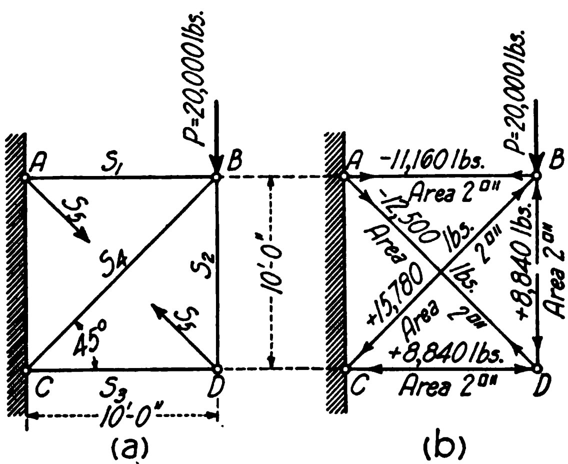

FIG. 60.

Solution.—Replace the member AD by the stress in the member S_5, which will be calculated later. Then calculate the stresses in the remaining members in terms of the stress in S_5. The stresses in the members are given in Table XIII, column 4. The derivative of the stress in each member with reference to S_5 is given in column 5. The deformation in each member for a unit stress, is $B = L/A \cdot E$, where L = length of the member in in., A = area of cross-section of member in sq. in., and $E = 30{,}000{,}000$ = modulus of elasticity of steel. Now if

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dS_5 is assumed as unity, then $\frac{dS}{dS_5}=U$, and the stresses in column 5 are the stresses in the members due to a stress of unity acting in place of S_5. The derivative of the work of each member with reference to S_5, will be $=B\cdot S\cdot\left(\frac{dS}{dS_5}\right)=\Delta y_1=$ the deformation of the structure in line of member S_5, due to stress of S.

The sum of the values in column 7 is

$$\frac{dW}{dS_5}=\sum B\cdot S\left(\frac{dS}{dS_5}\right)=y_1=\frac{2.707}{500{,}000}P+\frac{2.914}{500{,}000}S_5 \qquad (26)$$

Now the deformation of S_5 for a stress of S_5 lbs. is

$$\frac{1.414}{500{,}000}S_5=-y_1 \qquad (27)$$

Solving equations (26) and (27) for S_5 gives

$$S_5=-0.625P=-12{,}500 \text{ lbs.}$$

TABLE XIII.

CALCULATION OF WORK EQUATIONS FOR $P=20{,}000$ LBS.

1	2	3	4	5	6	7
Member.	Area, A, Sq. in.	Length, L, In.	Stress S, in Terms of S_5.	$\frac{dS}{dS_5}=U$.	$\frac{L}{A\cdot E}=B$	$B\cdot S\left(\frac{dS}{dS_5}\right)$
S_1	2.00	120	$P+0.707S_5$	$+0.707$	$\frac{1}{500{,}000}$	$\frac{0.707}{500{,}000}(P+0.707S_5)$
S_2	2.00	120	$-0.707S_5$	-0.707	$\frac{1}{500{,}000}$	$\frac{0.500}{500{,}000}S_5$
S_3	2.00	120	$-0.707S_5$	-0.707	$\frac{1}{500{,}000}$	$\frac{0.500}{500{,}000}S_5$
S_4	2.00	170	$1.414P+S_5$	$+1.000$	$\frac{1.414}{500{,}000}$	$\frac{2.00}{500{,}000}(P+0.707S_5)$
			$\frac{dW}{dS_5}=\sum B\cdot S\left(\frac{dS}{dS_5}\right)=y_1=$			$\frac{2.707}{500{,}000}P+\frac{2.914}{500{,}000}S_5$
S_5	2.00	170	S_5	$+1.00$	$\frac{1.414}{500{,}000}$	$-y_1=\frac{1.414}{500{,}000}S_5$

$$\frac{2.707}{500{,}000}P+\frac{2.914}{500{,}000}S_5+\frac{1.414}{500{,}000}S_5=0;\ S_5=-0.625P=-12{,}500 \text{ lbs.}$$

If member S_2 is rigid the second value in column 7 is zero, and

$$\frac{2.707}{500{,}000}P+\frac{2.214}{500{,}000}S_5+\frac{1.414}{500{,}000}S_5=0;\ S_5=-0.707P=-14{,}140 \text{ lbs.}=-S_4.$$

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If the member S_2 is rigid the second value in column 7 is zero, and solving as above

$$S_5 = -0.707P = -14,140 \text{ lbs.} = -S_4,$$

stresses S_4 and S_5 being equal, as they should be.

STRESSES IN A FRAMEWORK WITH TWO REDUNDANT MEMBERS.—The framework in Fig. 61 would be stable with either of the diagonal members in each panel, and therefore has two redundant members. The framework is statically determinate for external forces; $R_1 = 20,000$ lbs., and $R_2 = 10,000$ lbs.

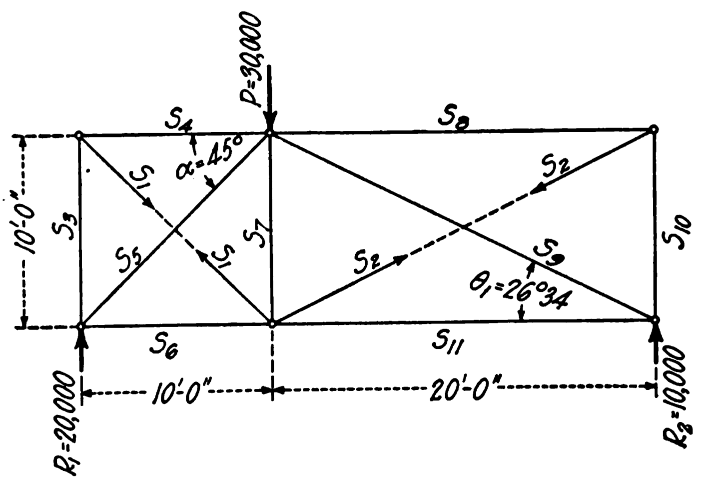

FIG. 61.

To calculate the stresses in the redundant members, replace the truss members by stresses S_1 and S_2, respectively, and solve for the unknown stresses. In Table XIV the stresses in the members of the framework in terms of S_1 and S_2 are given in column 4; $dS/dS_1 = U$, and $dS/dS_2 = U'$ are given in columns 5 and 6, respectively; values of $L/A \cdot E = B$ are given in column 7; values of $B \cdot S(dS/dS_1)$ in column 8, and $B \cdot S(dS/dS_2)$ in column 9. Now the sum of column 8 is equal to the displacement in line of member $S_1 = y_1$; also $y_1 = -\frac{2.83}{500,000} S_1$.

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TABLE XIV.

Calculation of Work Equations for $P = 30{,}000$ Lbs.

1	2	3	4	5	6	7	8	9
Member.	Area, A, Sq. In.	Length, L, In.	Stress S in terms of S_1 and S_2.	$\frac{d\cdot S}{dS_1} = U$	$\frac{dS}{dS_2} = U'$	$\frac{L}{A\cdot E} = B$	$B\cdot S\left(\frac{dS}{dS_1}\right)$	$B\cdot S\left(\frac{dS}{dS_2}\right)$
S_3	2.00	120	$-0.707S_1$	-0.707		$\frac{1}{500{,}000}$	$+\frac{0.500}{500{,}000}S_1$	
S_4	2.00	120	$-0.707S_1$	-0.707		$\frac{1}{500{,}000}$	$+\frac{0.500}{500{,}000}S_1$	
S_5	1.00	170	$-28{,}280 - S_1$	-1.00		$\frac{2.83}{500{,}000}$	$+\frac{80{,}000}{500{,}000}+\frac{2.83}{500{,}000}S_1$	
S_6	1.00	120	$-20{,}000 - 0.707S_1$	-0.707		$\frac{2}{500{,}000}$	$+\frac{20{,}000}{500{,}000}+\frac{1.00}{500{,}000}S_1$	
S_7	2.00	120	$-0.707S_1 - 0.448S_2$	-0.707	-0.448	$\frac{1}{500{,}000}$	$+\frac{0.500}{500{,}000}S_1+\frac{0.317}{500{,}000}S_2$	$+\frac{0.317}{500{,}000}S_1+\frac{0.200}{500{,}000}S_2$
S_8	2.00	240	$-0.895S_2$		-0.895	$\frac{2}{500{,}000}$		$+\frac{1.600}{500{,}000}S_2$
S_9	1.00	268	$-22.333 - S_2$		-1.00	$\frac{4.47}{500{,}000}$		$+\frac{99{,}800}{500{,}000}+\frac{4.47}{500{,}000}S_2$
S_{10}	2.00	120	$-0.448S_2$		-0.448	$\frac{1}{500{,}000}$		$+\frac{0.200}{500{,}000}S_2$
S_{11}	1.00	240	$-20{,}000 - 0.895S_2$		-0.895	$\frac{4}{500{,}000}$		$+\frac{71\,600}{500{,}000}+\frac{3.20}{500{,}000}S_2$
							$y_1 = +\frac{100{,}000}{500{,}000}+\frac{5.33}{500{,}000}S_1+\frac{0.317}{500{,}000}S_2$	$y_2 = +\frac{171{,}400}{500{,}000}+\frac{0.317}{500{,}000}S_1+\frac{9.67}{500{,}000}S_2$
S_1	1.00	170	S_1	-1.00		$\frac{2.83}{500{,}000}$	$-\frac{2.83}{500{,}000}S_1 = -y_1$	
S_2	1.00	268	S_2		-1.00	$\frac{4.47}{500{,}000}$		$-\frac{4.47}{500{,}000}S_2 = -y_2$

$$(1)\quad 8.160S_1 + 0.317S_2 = -100{,}000$$
$$(2)\quad 0.317S_1 + 14.140S_2 = -171{,}400$$
$$(3)\quad 370{,}000S_1 + 14.140S_2 = -4{,}500{,}000$$
$$(4)\quad 369{,}683S_1 = -4{,}328{,}400$$
$$S_1 = -11{,}800 \text{ lbs.}$$
$$S_2 = -11{,}400 \text{ lbs.}$$

The sum of column 9 is equal to the displacement in line of member $S_2 = y_2$; also $y_2 = -\frac{4.47}{500,000} S_2$. Equating the values of y_1 and y_2, and solving we have

$$8.160\,S_1 + 0.317\,S_2 = -100,000 \tag{28}$$

$$0.317\,S_1 + 14.140\,S_2 = -171,400 \tag{29}$$

Multiply (28) by 14.14/0.317, and

$$370,000\,S_1 + 14.140\,S_2 = -4,500,000 \tag{30}$$

and subtracting (29) from (30) we have

$$369,683\,S_1 = -4,328,400 \text{ lbs.}$$
$$S_1 = -11,800 \text{ lbs}$$

also

$$S_2 = -11.400 \text{ lbs.}$$

Substituting in column 4, Table XIV, gives $S_8 = -16,480$ lbs., and $S_9 = -10,933$ lbs.

WORK OF FLEXURE.—Let (*a*), Fig. 62, be a portion of a beam which is in flexure under the action of an external load or loads,

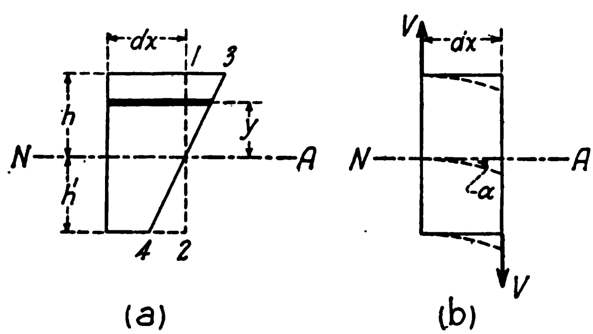

FIG. 62.

and has a bending moment M at the section *1–2*. Then the stress in any fiber at a distance y from the neutral axis NA is $S' = \frac{M \cdot y}{I}$; and

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the deformation of any differential length dx at a distance y from the neutral axis NA will be

$$d\Delta = \frac{S'}{E} dx = \frac{M}{E \cdot I} y \cdot dx \tag{31}$$

where E is the modulus of elasticity of the material composing the beam, and I is the moment of inertia of the section *1–2* about the neutral axis NA. Then the deformation of a fiber of length dx at the outer fiber will be

$$d\Delta' = \frac{M}{E \cdot I} h \cdot dx \tag{32}$$

If z is the breadth of the beam at a distance y from NA, then the stress on a differential area dA of the fiber at a distance y from NA will be

$$S = \frac{M}{I} z \cdot y \cdot dy \tag{33}$$

and the work of resistance of the differential fiber will be

$$dw = \frac{1}{2} \frac{M}{E \cdot I} y \cdot dx \times \frac{M}{I} z \cdot y \cdot dy \tag{34}$$

Integrating (34) between the limits $+h$ and $-h'$, the work of resistance on the length of beam dx, is

$$dW = \frac{1}{2} \frac{M^2}{E \cdot I^2} dx \int_{-h'}^{+h} z \cdot y^2 dy \tag{35}$$

Now

$$\int_{-h'}^{+h} z \cdot y^2 dy = I, \quad \text{and} \quad dW = \frac{1}{2} \frac{M^2}{E \cdot I} dx \tag{36}$$

The total work of resistance in a beam with a length l will be

$$W = \int_0^l \frac{M^2}{2E \cdot I} dx \tag{37}$$

Now

$$y_1 = \frac{dW}{dP} = \frac{d \int_0^l \frac{M^2}{2E \cdot I} dx}{dP} = \int_0^l M \left(\frac{dM}{dP} \right) \frac{dx}{E \cdot I} \tag{38}$$

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Now if $P = 1$, $\frac{dM}{dP}$ will be the bending moment at the point due to a load unity $= M'$, and

$$y_1 = \int_0^l M \cdot M' \frac{dx}{E \cdot I} \tag{39}$$

To calculate the deflection in a beam with a load P at a distance a from the left abutment, it is more convenient to write equation (37) in the form, where E and I are constant

$$W = \frac{1}{2E \cdot I} \int_0^a M_1^2 \cdot dx + \frac{1}{2E \cdot I} \int_a^l M_2^2 \cdot dx \tag{40}$$

where M_1 is the bending moment in the beam at any point to the left of the load, and M_2 is the bending moment at any point to the right of the load.

WORK OF SHEAR.—The work of resistance due to shear is usually so small that it may be neglected. In (b), Fig. 62, let V be the shear on the section, A = area of the cross-section, and G = modulus of elasticity in shear.

Now the angular distortion due to shear will be

$$a = \frac{V}{G \cdot A} \tag{41}$$

and the work due to shear in a distance dx, if the shear is uniformly distributed over the section, will be

$$dW_s' = \tfrac{1}{2} V \cdot a \cdot dx = \frac{V^2}{2G \cdot A} dx$$

But the shear is not in general uniformly distributed, and

$$dW_s = \frac{n \cdot V^2}{2G \cdot A} dx \tag{42}$$

Then the work due to shear is

$$W_s = \int_0^l \frac{n \cdot V^2}{2G \cdot A} dx \tag{43}$$

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where

$$n=\frac{A}{V^2}\int_{-h'}^{h} t^2\cdot dA,$$

where

$$t=\frac{V}{b\cdot I}\int_{h''}^{h} y\cdot dA,$$

where h'' = distance above NA to the fiber for which t is to be found, b = width of the cross-section, y = ordinate of the given point above NA, and I = static moment of inertia of the cross-section with respect to the neutral axis; n is a quantity greater than 1 and less than 2. For a rectangular beam, $n=\frac{6}{5}$; for a solid circular beam, $n=\frac{10}{9}$; while for a steel I-beam, $n=1$ (approximately).

DEFLECTION OF A BEAM.—The internal work in a simple beam with a constant cross-section, having a span l, and carrying a load P at the middle will be

$$W=\int_0^l \frac{M^2}{2E\cdot I}dx+\int_0^l \frac{n\cdot V^2}{2G\cdot A}dx \tag{44}$$

$$=\frac{1}{2E\cdot I}\left[\int_0^{l/2} M_1^2\cdot dx+\int_{l/2}^{l} M_2^2\cdot dx\right]+\int_0^l \frac{n\cdot V^2}{2G\cdot A}dx \tag{45}$$

Now $M_1=\frac{1}{2}P\cdot x$, $M_2=\frac{1}{2}P(l-x)$, and $V=P/2$

$$W=\frac{1}{8E\cdot I}\left[\int_0^{l/2} P^2\cdot x^2\cdot dx+\int_{l/2}^{l} P^2\cdot l^2\cdot dx-\int_{l/2}^{l} 2P^2\cdot l\cdot x\cdot dx\right.$$

$$\left.+\int_{l/2}^{l} P^2\cdot x^2\cdot dx\right]+\int_0^l \frac{1}{8}\frac{n\cdot P^2}{G\cdot A}dx \tag{46}$$

$$=\frac{P^2\cdot l^3}{96E\cdot I}+\frac{n\cdot P^2\cdot l}{8G\cdot A} \tag{47}$$

But

$$\frac{dW}{dP}=y_1,$$

and

$$y_1=\frac{1}{48}\frac{P\cdot l^3}{E\cdot I}+\frac{n\cdot P\cdot l}{4G\cdot A} \tag{48}$$

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For a 12-inch steel I-beam @ $31\frac{1}{2}$ lbs., 20 ft. long, and $P = 10{,}000$ lbs., $n = 1$, $E = 30{,}000{,}000$ lbs. per sq. in., and $G = 12{,}000{,}000$ lbs. per sq. in., $I = 215.8$ in.^{4}, area of web, $A = 12 \times 0.35 = 4.20$ sq. in.

Then

$$y_1 = 0.089 \text{ ins.} + 0.00119 \text{ ins.}$$

The deflection due to shear is then less than $1\frac{1}{2}$ per cent of that due to bending moment.

Deflection of a Simple Beam with a Uniform Load.—A simple beam with span l, carries a uniform load w per lineal unit of length.

Now

$$y_1 = \int_0^l M \cdot M' \frac{dx}{E \cdot I} \tag{49}$$

where for a beam uniformly loaded, M is the bending moment at any point due to the loading, and M' is the bending moment due to a load unity placed at the point at which the deflection is to be calculated. Now at any point x measured from the left end, $M = \frac{w \cdot l \cdot x}{2} - \frac{w \cdot x^2}{2}$, and $M' = \frac{(l - a)x}{l}$ on the left of the point, and $\frac{(l - x)a}{l}$ on the right of the point, where a is the distance of the point from the left end. Substituting in equation (49) gives

$$y_1 = \int_0^l \left(\frac{w \cdot l \cdot x}{2} - \frac{w \cdot x^2}{2} \right) M' \frac{dx}{E \cdot I} \tag{50}$$

Now for the center of the beam $M' = \frac{x}{2}$, and

$$y_1 = 2 \int_0^{l/2} \left(\frac{w \cdot l \cdot x}{2} - \frac{w \cdot x^2}{2} \right) \frac{x}{2} \frac{dx}{E \cdot I} \tag{51}$$

$$= \frac{w}{EI} \left[\frac{l \cdot x^3}{6} - \frac{x^4}{8} \right]_0^{l/2} \tag{52}$$

$$= \frac{5}{384} \frac{w \cdot l^4}{E \cdot I} \tag{53}$$

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DEFLECTION AT CENTER OF A BEAM FIXED AT BOTH ENDS, Concentrated Load at Middle.—A beam of constant cross-section carrying a load, P, at the center is fixed at both ends. It is required to calculate the deflection at the center.

Let M_0 = moment at left support and M_1 = moment at any point for a simple beam, then the bending moment at any point will be $M = M_0 + M_1$.

At left of center $M = M_0 + \frac{P \cdot x}{2}$, and at right of center

$$M = M_0 + \frac{P \cdot x}{2} - P\left(x - \frac{l}{2}\right) \tag{54}$$

Now the deflection at the left support is zero, and $M' = x$, for a unit load at left support. Then

$$y_e = 0 = \frac{2}{E \cdot I}\int_0^{l/2}\left(M_0 \cdot x + \frac{P \cdot x^2}{2}\right)dx$$

$$+ \frac{2}{E \cdot I}\int_{l/2}^{l}\left(M_0 \cdot x - \frac{P \cdot x^2}{2} + \frac{P \cdot l \cdot x}{2}\right)dx \tag{55}$$

from which $M_0 = -\frac{P \cdot l}{8}$ (56)

Then to calculate the center deflection

$$M = M_0 + \frac{P \cdot x}{2} = -\frac{P \cdot l}{8} + \frac{P \cdot x}{2}$$

and
$$M' = -\frac{l}{8} + \frac{x}{2}$$

and substituting in

$$y_c = \frac{2}{E \cdot I}\int_0^{l/2} M \cdot M' dx$$

$$y_c = \frac{P}{2E \cdot I}\int_0^{l/2}\left[\frac{l^2}{16} - \frac{l \cdot x}{2} + x^2\right]dx \tag{57}$$

$$= \frac{P}{2E \cdot I}\left[\frac{l^3}{32} - \frac{l^3}{16} + \frac{l^3}{24}\right]$$

$$= \frac{P \cdot l^3}{192E \cdot I} \tag{58}$$

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Deflection at Center of a Beam Fixed at Both Ends, Uniform Load.—A beam of constant cross-section carrying a load, w, per lineal foot, is fixed at both ends. It is required to calculate the deflection at the center.

Let M_0 = moment at the left support, then the bending moment at any point will be $M = M_0 + M_1$, where M_0 and M_1 are as in the preceding problem, and

$$M = M_0 + \frac{w \cdot l \cdot x}{2} - \frac{w \cdot x^2}{2}$$

Now the deflection at the left support is zero. Then with a load unity at the left support $M' = x$, and

$$y_e = 0 = \frac{1}{E \cdot I} \int_0^l \left(M_0 \cdot x + \frac{w \cdot l \cdot x^2}{2} - \frac{w \cdot x^2}{2} \right) dx$$

$$= \left[\frac{M_0 \cdot x^2}{2} + \frac{w \cdot l \cdot x^2}{2} - w \cdot x^3 \right]_0^l = 0 \qquad (59)$$

and

$$M_0 = - \frac{w \cdot l^2}{12} \qquad (60)$$

The moment at any point is then

$$M = - \frac{w \cdot l^2}{12} + \frac{w \cdot l \cdot x}{2} - \frac{w \cdot x^2}{2} \qquad (61)$$

Now from the preceding problem with a load unity at the center

$$M' = - \frac{l}{8} + \frac{x}{2}$$

and substituting in

$$y_c = \frac{2}{E \cdot I} \int_0^{l/2} M \cdot M' dx$$

$$y_c = \frac{w}{2E \cdot I} \int_0^{l/2} \left(\frac{l^3}{24} - \frac{10l^2 \cdot x}{24} + \frac{30l^2 \cdot x}{24} - x^3 \right) dx \qquad (62)$$

$$= \frac{w}{2E \cdot I} \left[\frac{l^4}{48} - \frac{5l^4}{96} + \frac{10l^4}{192} - \frac{l^4}{64} \right]$$

$$= \frac{w \cdot l^4}{384E \cdot I} \qquad (63)$$

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For other applications of the Theory of Least Work see Hiroi's "Statically Indeterminate Stresses," Church's "Mechanics of Internal Work," and Johnson, Bryan and Turneaure's "Modern Framed Structures," Part II.

METHOD OF AREA MOMENTS.—The statement of the fundamental theorem of area moments is: *If A and B are two points in a beam, the deflection of B with respect to a tangent at A, is equal to the moment of the area of the portion of the equilibrium polygon between A and B, multiplied by $H/E \cdot I$ where E is the modulus of elasticity of the material, I is the moment of inertia of the section of the beam, and H is the pole distance of the force polygon that was used in drawing the equilibrium polygon.*

Proof.—Now from (39) the deflection of B with respect to a tangent at A is

$$y_1 = \int_A^B M \cdot M' \frac{dx}{E \cdot I} \tag{39}$$

Now if a unit load is placed at B, the bending moment at a distance x from B will be, $M' = x$. Also let an equilibrium polygon be drawn for the given loading with a force polygon having a pole distance H; and if $z =$ the ordinate to the equilibrium polygon at any point, then $M = H \cdot z$ (see "The Design of Steel Mill Buildings," Chapter V). Substituting the values of M and M' in (39)

$$y_1 = H \int_A^B z \cdot x \frac{dx}{E \cdot I} \tag{64}$$

But $z \cdot x$ is the moment of the bending moment at any point about the point B, and (64) becomes

$$y_1 = H \frac{(\textit{Moment of Area equilibrium polygon A to B about B})}{E \cdot I} \tag{65}$$

which proves the principle of area moments. (See Chapter XVA, "The Design of Steel Mill Buildings," for a direct proof not using the

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work equation.) The application of the method of area moments will be illustrated by means of several problems.

Deflections of Simple Beams.—The simple beam will be considered when loaded with concentrated and with uniform loads, using both algebraic and graphic methods.

Algebraic Method—Concentrated Load at Center of Beam.—The simple beam in (a), Fig. 63, is loaded with a load, P, at the center. The bending moment diagram is shown in (b), and the beam is loaded with the bending moment diagram in (c), Fig. 63.

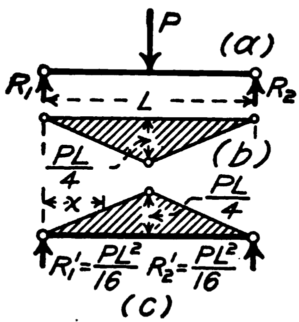

FIG. 63.

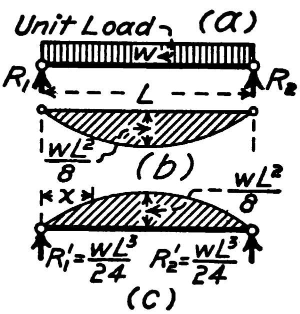

FIG. 64.

To find the equation of the elastic curve, take moments of the forces to the left of a point at a distance x from the left support, and

$$-E\cdot I\cdot y=\frac{P\cdot L^2\cdot x}{16}-\frac{P\cdot x^4}{12},$$

and

$$48E\cdot I\cdot y=P(4x^3-3L^2\cdot x) \qquad (66)$$

The maximum deflection will occur when $x=\frac{1}{2}L$ in (66), or it may be found by taking moments of forces to left of $x=\frac{1}{2}L$, to be

$$\Delta=\frac{P\cdot L^3}{48E\cdot I} \qquad (66')$$

Beam Uniformly Loaded.—The simple beam in (a), Fig. 64, is loaded with a uniform load of, w, per lineal foot. The bending moment

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parabola is shown in (b), and the beam is loaded with the bending moment parabola in (c), Fig. 64. To find the equation of the elastic curve, take moments of forces to the left of a point at a distance x from the left support.

The equation of the bending moment parabola, with the origin of coördinates at the left support, is $y = \frac{1}{2}P\cdot L\cdot x - \frac{1}{2}P\cdot x^2$; the area of a segment of the parabola, is $A = \frac{1}{4}w\cdot L\cdot x^2 - 1\text{–}6\, w\cdot x^3$, and the center of gravity measured back from x is

$$-X = \frac{x(2L - x)}{6L - 4x}$$

Taking moments of forces to the left of a point x, and reducing, we have

$$24\, E\cdot I\cdot y = w(-x^4 + 2L\cdot x^3 - L^2\cdot x) \qquad (67)$$

The deflection is a maximum when $x = \frac{1}{2}L$, and may be found directly by taking moments, or may be found from (67), and is

$$\Delta = \frac{5}{384}\,\frac{w\cdot L^4}{E\cdot I} \qquad (68)$$

Cantilever Beam—Concentrated Load.—The cantilever beam in (a), Fig. 65, has a concentrated load, P, at its extreme end. It will be

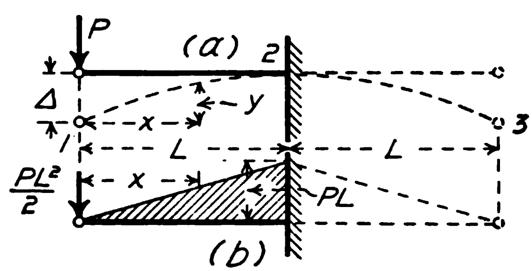

FIG. 65.

seen that the cantilever beam may be considered as one half of a simple beam with a span, $2L$, and a load, $2P$, at the center. The equation of the elastic curve may be found as in Fig. 63. Load the beam with the

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bending moment diagram as in (*b*), Fig. 65, and considering the cantilever as one half of the simple beam, we have, after reducing,

$$6E \cdot I \cdot y = 3P \cdot L^2 \cdot x - P \cdot x^3 \qquad (69)$$

The maximum value of Δ is equal to y when x equals L, and

$$\Delta = \frac{P \cdot L^3}{3E \cdot I} \qquad (70)$$

To find the maximum deflection take the moment of the entire bending moment parabola about the point 1, and

$$E \cdot I \cdot \Delta = \frac{P \cdot L^2}{2} \times \tfrac{2}{3}L$$

$$\Delta = \frac{P \cdot L^3}{3E \cdot I}$$

This method of finding the maximum deflection of a cantilever beam is the one to use in calculations, and will be used in the solution of the problem of the transverse bent.

Deflection of a Simple Beam. Graphic Method.*—It is proved in books on applied mechanics that the differential equation of the elastic curve of a beam is

$$\frac{d^2y}{dx^2} = \frac{M}{E \cdot I} \qquad (71)$$

It has been proved in equation (65) and in "The Design of Steel Mill Buildings" that if a beam be loaded with a continuous load represented by the equation $y' = fx$, that a force polygon be constructed with a pole distance H, and that an equilibrium polygon be constructed using this force polygon, the differential equation of the equilibrium polygon is

$$\frac{d^2y}{dx^2} = \frac{fx}{H} \qquad (72)$$

By comparing equations (71) and (72) it will be seen that if fx

* For the details of drawing a force polygon and its equilibrium polygon, see the author's "The Design of Steel Mill Buildings," Chapter V.

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and H, in (72), be taken equal to M and $E \cdot I$, respectively, in (71), the equilibrium polygon will be the elastic curve of the beam.

To construct the elastic curve of a beam with a constant moment of inertia I, and constant modulus of elasticity E, proceed as follows:

Construct the bending moment polygon for the given loading on the beam. Load the beam with this bending moment polygon, and with a force polygon having a pole distance, $E \cdot I$, construct an equilibrium polygon; this polygon will be the elastic curve of the beam. It is not commonly convenient to use a pole distance equal to $E \cdot I$, and a pole distance H is used, where $N \cdot H$ equals $E \cdot I$; the deflection at any point will then be equal to the measured deflection divided by N.

If the moment of inertia is variable, we may take $fx = M/I$, and use a pole distance equal to E. The beam will then be loaded with a load polygon the ordinate at any point of which is $y' = M/I$. If E is also a variable the beam may be loaded with a load, $fx = \frac{N \cdot M}{E \cdot I}$, and the pole distance will be $H = N$.

It will thus be seen that the deflection of a simple beam of variable moment of inertia, loaded with any system of loads, may be calculated with comparative ease.

Simple Beam.—In Fig. 66 a simple beam is loaded with a load, P, as shown. With force polygon (b), draw equilibrium polygon (c). Now load the beam with equilibrium polygon as in (c), and divide the area of the equilibrium polygon into segments, which are treated as loads acting through their centers of gravity. Construct force polygon (d) and draw equilibrium polygon (e).

Now the deflection at any point having an ordinate y, in (e), will be, if proper scales are used,

$$\Delta = \frac{y \times H \times H'}{E \cdot I}$$

In Fig. 66, if P equals 3,000 lbs., and the area of the equilibrium polygon and pole distance H' are measured in square-foot pounds, pole distance H in pounds, and y in feet, we will have

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$$\Delta = \frac{y \times H \times H' \times 1728}{E \cdot I}$$

$= 1.88$ in. at center, while the maximum value of deflection is

$$\Delta' = 1.92 \text{ in.}$$

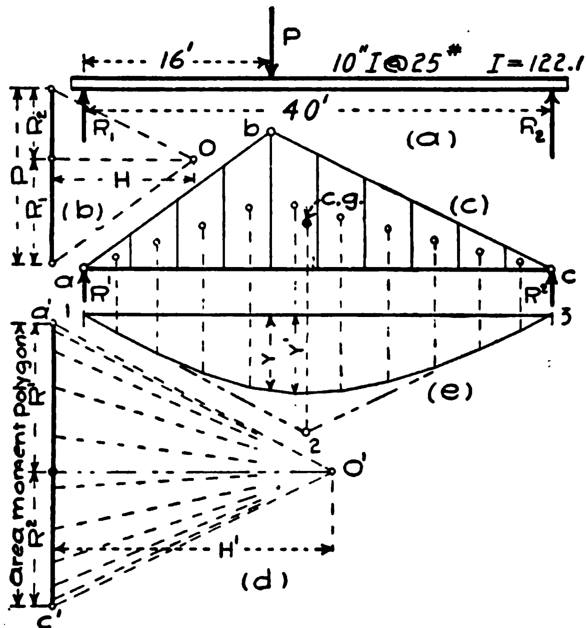

FIG. 66.

Tangents to Elastic Curve.—If strings 1 and 3 in (*e*), Fig. 66, be produced, they will intersect at 2, on a line through the center of gravity of the moment-area polygon, and the strings 1–2 and 2–3 will be tangents to the elastic curve at the supports R_1 and R_2, respectively. This gives an easy method for constructing the tangents to the elastic curve without constructing the curve. It is also seen that the tangents to the elastic curve depend only on the amount of the moment area and the position of its center of gravity, and are independent of the arrangement of the moment areas.

Deflection of a Beam with Variable Moment of Inertia.—A simple beam with a variable moment of inertia, and having a length l,

carries a uniform load w, per lineal foot of beam. The moment of inertia of the middle half of the beam is I_2, while the moment of inertia of the end quarters is I_1. Required to calculate the maximum deflection of the beam.

Load the outer quarters of the beam in Fig. 67 with $fx = M/I_1$, and the middle half with $fx = M/I_2$.

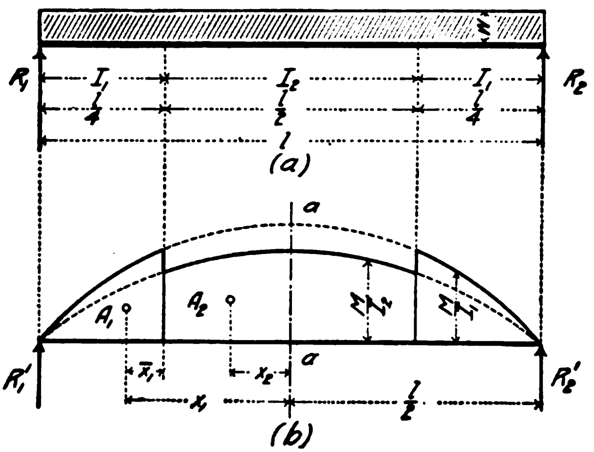

FIG. 67.

Now the area of the segment of the parabola A_1, in which x is the distance from the left support to the end of the segment, is

$$A_1 = \frac{1}{4}\frac{w \cdot l \cdot x^2}{I_1} - \frac{1}{6}\frac{w \cdot x^3}{I_1} = \frac{5}{384}\frac{w \cdot l^3}{I_1} \tag{73}$$

Area A_2 equals the area of the half parabola, whose ordinates are $fx = M/I_2$, minus the area of the outer quarter of the parabola, and

$$A_2 = \frac{1}{24}\frac{w \cdot l^3}{I} - \frac{5}{384}\frac{w \cdot l^3}{I_2} = \frac{11}{384}\frac{w \cdot l^3}{I_2} \tag{74}$$

Now the distance x_1 from the center of the beam to the center of gravity A_1, is

$$x_1 = \bar{x}_1 + \frac{l}{4}$$

where

$$\bar{x}_1 = \frac{x(2l - x)}{6l - 4x} = \frac{7l}{80} \tag{75}$$

and

$$x_1 = \frac{27l}{80}$$

To calculate x_2, subtract the static moment about $a - a$, of the left quarter of the parabola where $fx = M/I_2$, from the static moment about $a - a$ of the left half of the same parabola, and divide by the area A_2. The center of gravity of A_2 is

$$x_2 = \frac{\left[\frac{1}{24}\frac{w \cdot l^3}{I_2} \times \frac{8}{16}l - \frac{5}{384}\frac{w \cdot l^3}{I_2} \times \frac{27}{80}l\right]}{\frac{11}{384}\frac{w \cdot l^3}{I_2}} = \frac{21}{176}l \tag{76}$$

Now

$$R_1' = R_2' = A_1 + A_2 = \frac{5}{384}\frac{w \cdot l^3}{I_1} + \frac{11}{384}\frac{w \cdot l^3}{I_2}$$

Now the maximum deflection Δ will be found at the center of the beam, and will be equal to the bending moment at the center of the beam divided by E, and

$$E \cdot \Delta = R_1' \times \frac{l}{2} - A_1 \times x_1 - A_2 \times x_2$$

$$= w \cdot l^3\left[\frac{5}{768}\frac{l}{I_1} + \frac{11}{768}\frac{l}{I_2} - \frac{27}{6144}\frac{l}{I_1} - \frac{21}{6144}\frac{l}{I_2}\right] \tag{77}$$

Reducing and dividing by E

$$\Delta = \frac{w \cdot l^4}{I_1 \cdot I_2 \cdot E}\left[\tfrac{13}{6144}I_2 + \tfrac{67}{6144}I_1\right]$$

Deflection of Beams Fixed at Ends.—The simple beam in Fig. 66 is loaded with a load P. The beam is loaded with the bending polygon (c), calculated by means of force polygon (b), and equilibrium polygon (e) is calculated by means of force polygon (d). The ordinates of equilibrium polygon (e) will be proportional to the true deflections.

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From Fig. 66 it is seen that the tangents 1–2 and 2–3 to the elastic curve at the supports depend upon the amount of the bending moment and the position of the center of gravity of the moment area, and are independent of the arrangement of the moment. To fix the ends of the beam in (*a*), Fig. 68, it will therefore be necessary to have a negative bending moment equal to the positive bending moment in amount, and having its center of gravity on the same vertical line.

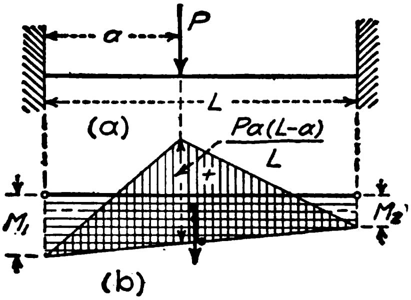

FIG. 68.

It therefore follows that for a straight beam, with constant moment of inertia, I, and modulus of elasticity E, to have fixed ends the following equations of condition must be satisfied

$$\Sigma M = 0 \tag{78}$$

$$\Sigma M \cdot x = 0 \tag{79}$$

In Fig. 68 a beam of span L, fixed at the ends is loaded with a load, P, at a distance a from the left end. The bending moment at the left and right ends is M_1 and M_2, respectively, M_1 and M_2 being negative, and the moment under the load is $M = P \cdot a(L - a)/L$. To calculate M_1 and M_2, substitute in equations (78) and (79), and solve for the unknown bending moments.

From equation (78)

$$\frac{P \cdot a}{2}(L - a) + \frac{(M_1 + M_2)L}{2} = 0 \tag{78'}$$

and from equation (79), taking moments about the left end of the beam,

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$$\frac{P \cdot a^2(L-a)}{2L} \times \tfrac{2}{3}a + \frac{P \cdot a(L-a)^2}{2L} \times \left(a + \frac{L-a}{3}\right) + \frac{M_1 \cdot L}{2} \times \frac{L}{3}$$
$$+ \frac{M_2 \cdot L}{2} \times \frac{2L}{3} = 0 \qquad (79')$$

The values of M_1 and M_2 can be calculated from equations (78′) and (79′) for any value of a.

Now if $a = L/2$, we have, after reducing

$$\frac{P \cdot L^2}{2} + M_1 \cdot L + M_2 \cdot L = 0 \qquad (78'')$$

And

$$\tfrac{5}{48} P \cdot L^3 + \frac{M_1 \cdot L^2}{6} + \frac{M_2 \cdot L^2}{3} = 0 \qquad (79'')$$

Multiply (78″) by $L/6$, and

$$\frac{P \cdot L^3}{12} + \frac{M_1 \cdot L^2}{6} + \frac{M_2 \cdot L^2}{6} = 0 \qquad (80)$$

Subtracting (80) from (79″), we have

$$\tfrac{1}{48} P \cdot L^3 + \frac{M_2 \cdot L^2}{6} = 0$$

and

$$M_2 = -\tfrac{1}{8} P \cdot L = M_1 \qquad (81)$$

The maximum moment at the center will be $\frac{1}{8} P \cdot L$, and the points of contra-flexure will be at the quarter points of the beam.

CONTINUOUS BEAMS.—A beam which in an unstrained condition rests on more than two supports is a continuous beam. For a straight beam the supports must all be on the same level. Beams of one span, with one end fixed and the other end supported, or with both ends fixed, may also be considered as continuous beams.

In Fig. 69 the continuous beam in (a) with spans L_1 and L_2, carries a uniform load, w, per lineal foot. It is required to calculate the reactions R_1, R_2, and R_3.

The reactions of the continuous beam in (a) may be replaced by

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the reactions of the two simple beams loaded with the uniform load, w, in (b), and the reactions and the load of the simple beam with the span $L_1 + L_2$, and carrying a negative load r' in (c). The reactions in (a) will then have the following values:

$$R_1 = r_1 - r_1'; \quad R_2 = r_2 + r_2'; \quad R_3 = r_3 - r_3'.$$

Now the upward curvature of the beam in (a), due to the load r_2', will be neutralized by the load above equal to r_2', which is transferred to the reaction R_2 by flexure in the beam. The upward deflection of

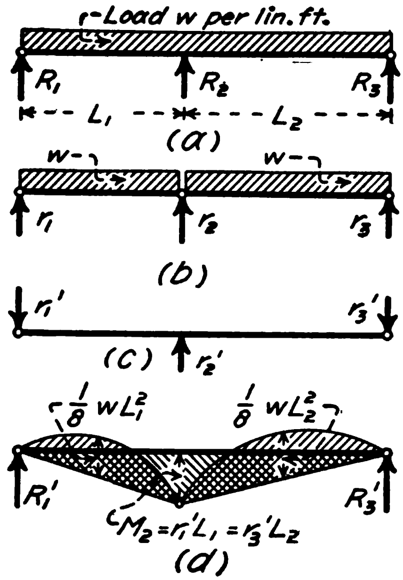

FIG. 69.

the beam in (c) at any point will be the bending moment divided by $E \cdot I$ at the same point in (d) due to a bending moment polygon with a maximum moment $M_2 = r_1' \times L_1 = r_3' \times L_2$; and the downward deflection of the beam in (b) at any point will be the bending moment divided by $E \cdot I$ at the same point in (d) due to the bending moment polygons for a uniform load w covering the simple spans in (b). But the deflection of the beam in (a) is zero at the reaction R_2, and therefore the bending moment at the corresponding point in (d) is zero.

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From the above discussion it follows that to calculate the reactions of the continuous beam in (*a*) by moment areas, take a simple beam with a span equal to $L_1 + L_2$, and load it with the bending moment polygons for beams (*b*) and (*c*) as in (*d*); the bending moment in beam (*d*) at the points corresponding to the reactions will be equal to zero, and the reactions of beam (*a*) can be calculated by statics when M_2 is obtained.

Continuous Beams—Concentrated Loads.—In (*a*), Fig. 70, a continuous beam of two equal spans of length, L, is loaded with two equal loads, P, at the centers of the spans. Calculate the bending moments

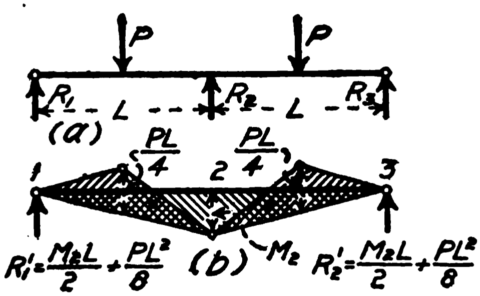

FIG. 70.

and load a simple beam with a span equal to $2L$, with the bending-moment diagrams due to P in each span, and with the negative bending-moment diagram due to the reaction R_2. Then to find M_2, the bending moment at 2, take moments of forces to the left of 2, and

$$\frac{M_2 \cdot L^2}{2} + \frac{P \cdot L^3}{8} - \frac{M_2 \cdot L^2}{6} - \frac{P \cdot L^3}{16} = 0,$$

and
$$M_2 = -\tfrac{3}{16} P \cdot L$$

To calculate R_1 take moments in (*a*) about 2, and

$$R_1 \cdot L - \frac{P \cdot L}{2} - M_2 = 0$$

$$R_1 = \tfrac{5}{16} P = R_3,$$

and
$$R_2 = \tfrac{11}{8} P$$

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Beam Uniformly Loaded.—In (*a*), Fig. 71, a continuous beam of three equal spans of length, L, is loaded with a uniform load equal to w per foot. Calculate the bending moments due to a uniform load of w on each span, and load a simple beam of span $3L$ with the positive

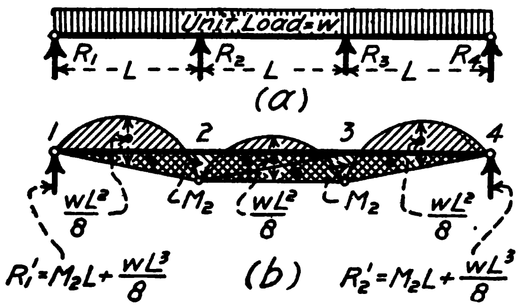

FIG. 71.

bending-moment diagrams due to load w, and with the negative bending-moment diagrams due to the reactions R_2 and R_3. The bending moment M_2 is equal to M_3. Now the deflection of the beam is zero at 2 and 3, and the bending moments must, therefore, be zero at these points. Taking moments of forces to the left of 2, we have

$$M_2 \cdot L^2 + \frac{w \cdot L^4}{8} - \frac{w \cdot L^4}{24} - \frac{M_2 \cdot L^2}{6} = 0$$

$$M_2 = -\frac{w \cdot L^2}{10} = M_3$$

To calculate R_1 take moments about 2 in (*a*), and

$$R_1 \cdot L - \frac{w \cdot L^2}{2} + M_2 = 0$$

$$R_1 = \tfrac{4}{10} w \cdot L = R_4$$

$$R_2 = R_3 = \tfrac{3}{2} w \cdot L - \tfrac{4}{10} w \cdot L = \tfrac{11}{10} w \cdot L$$

Continuous Beam of n Spans.—To calculate the reactions for a continuous beam of n spans, equal or unequal, loaded with any system or systems of loads, proceed as follows:

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Calculate the bending moment due to the external load, or loads, or system of loads, in each span considered as a simple beam. Take a simple beam having a total length equal to the length of the continuous beam, and load it with the bending moment polygons found as above. Also load the beam with the bending moment polygons due to the reactions. The reactions being unknown, the bending moments at the reactions will be unknown. Now calculate the bending moment in the simple beam, at points corresponding to each reaction, and place the result equal to zero, for the reason that the deflection at the supports is zero.

For a continuous beam of n spans there will be $n+1$ equations, which is equal to the number of unknown reactions. Solving these equations, the unknown moments will be found, and the reactions may be calculated algebraically.

COLUMNS OF A TRANSVERSE BENT.—The problem of the calculation of the point of contra-flexure in the columns of a transverse bent—the algebraic solution of which is given in Chapter XI of "The

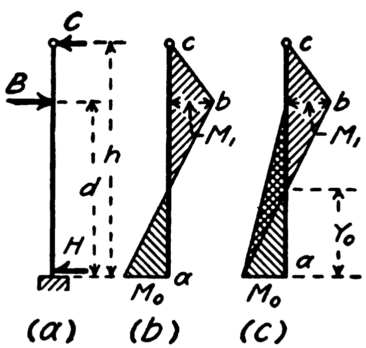

FIG. 72.

Design of Steel Mill Buildings"—will now be solved by the use of moment areas. The nomenclature in Fig. 72 is the same as in "The Design of Steel Mill Buildings." It is assumed that the deflection at points b and c are equal.

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In (b), Fig. 72, the deflection at b from the tangent at a is found by taking moments of the moment areas below b to be

$$E \cdot I \cdot \Delta = \frac{M_0 \cdot d}{2} \frac{2}{3} d - \frac{M_1 \cdot d}{2} \frac{d}{3}$$

$$\Delta = \frac{2M_0 \cdot d^2 - M_1 \cdot d^2}{6E \cdot I} \tag{82}$$

The deflection at c from the tangent at a is found by taking moments of moment areas below c to be

$$E \cdot I \cdot \Delta' = \frac{M_0 \cdot d}{2}\left(h - \frac{d}{3}\right) - \frac{2M_1(h-d)^2}{6} - \frac{M_1 \cdot d}{2}(h - \tfrac{2}{3}d)$$

$$\Delta' = \frac{M_0(3d \cdot h - d^2) - M_1(2h^2 - h \cdot d)}{6E \cdot I} \tag{83}$$

But Δ is equal to Δ', by hypothesis, and equating (82) and (83), we have

$$2M_0 \cdot d^2 - M_1 \cdot d^2 = M_0(3d \cdot h - d^2) - M_1(2h^2 - h \cdot d)$$

transposing,

$$M_0(3h \cdot d - 3d^2) = M_1(2h^2 - h \cdot d - d^2) \tag{84}$$

Now in (c), Fig. 72, it will be seen that $M_0 : M_1 :: y_0 : d - y_0$, and

$$M_0(d - y_0) = M_1 \cdot y_0 \tag{85}$$

Solving (84) and (85) for y_0, we have

$$y_0 = \frac{d}{2} \frac{(2h + d)}{(h + 2d)} \tag{86}$$

which is the same value as was found by algebraic methods in "The Design of Steel Mill Buildings."

CONTINUOUS BEAMS WITH MOVING LOADS.—The preceding methods are not adapted to the solution of problems involving moving loads, as in draw bridges or cranes on continuous girders. The following method, which is an application of curved influence lines, is quite simple in theory and application, although requiring considerable labor in preparing the diagrams. The solution will first be explained, and a proof given later.

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Girder with Three Supports.—In Fig. 73 a continuous beam with spans L_1 and L_2 is loaded with concentrated moving loads represented by P_1 and P_2, as in (a).

In (b) load a simple beam having a span $L_1 + L_2$ with a bending-moment polygon due to the reaction R_2 (the value of R_2 is unknown, and any convenient load will do).

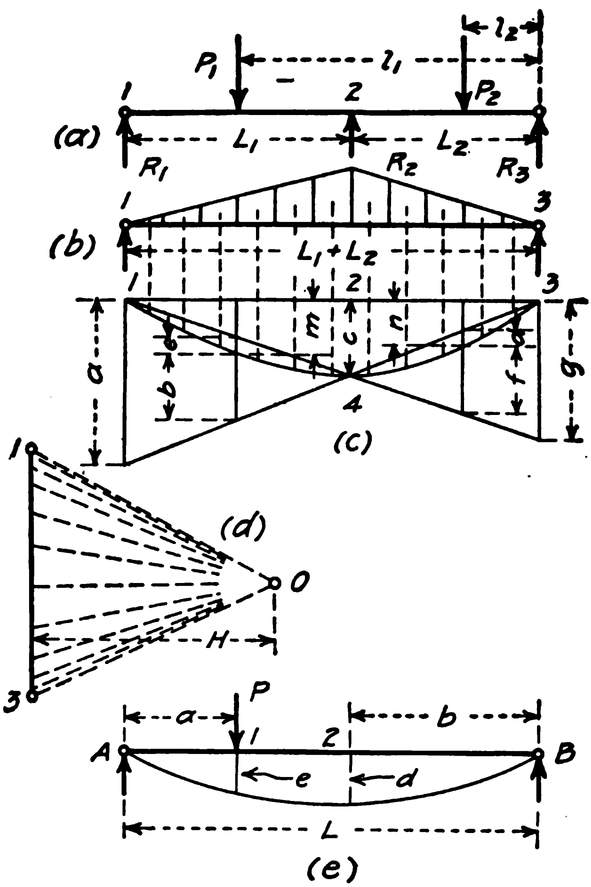

FIG. 73.

Divide the bending-moment diagram into segments, construct a force polygon as in (d), and draw an equilibrium polygon as in (c), Fig. 73,

assuming that the segments are loads acting through their centers of gravity.

The pole distance H may be taken as any convenient length, and the pole O may be taken at any point [in (c) the pole has been selected to bring the closing line horizontal, for convenience only].

Then in (c)

$$R_1 \cdot a = P_1 \cdot b - P_2 \cdot d$$

and

$$R_1 = \frac{P_1 \cdot b - P_2 \cdot d}{a} \tag{87}$$

$$R_2 \cdot c = P_1 \cdot m + P_2 \cdot n$$

and

$$R_2 = \frac{P_1 \cdot m + P_2 \cdot n}{c} \tag{88}$$

$$R_3 \cdot g = -P_1 \cdot e + P_2 \cdot f$$

and

$$R_3 = \frac{-P_1 \cdot e + P_2 \cdot f}{g} \tag{89}$$

Proof.—The ordinates to the equilibrium polygon in (c) are proportional to the ordinates to the true elastic curve of the beam in (b), when it is loaded with a given load at 2.

Now in (e), Fig. 73, if the deflection at 2 due to a load P at 1 is d, then if the load P be moved to 2, the deflection at 1 will be d. This is known as Maxwell's Theorem, and is proved as follows.

Maxwell's Theorem.—Maxwell's Theorem is "In a beam if a load P be placed at 1, and the deflection Δ_2 of the beam due to the load be measured at 2, (a), Fig. 74, then if the load P be placed at 2, (b), and the deflection Δ_1 be measured at 1, then $\Delta_1 = \Delta_2$."

Proof.—Let the load P be gradually applied at point 1 (c), Fig. 74, and the work on the beam will be $W_1 = \frac{1}{2}P \cdot \delta_1$ and the deflection at 2 will be Δ_2. Then if a load P be applied gradually at point 2, the deflec-

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tion at 1 will be Δ_1, (e), Fig. 74, and the work due to both loads will be

$$W = \tfrac{1}{2}P \cdot \delta_1 + P \cdot \Delta_1 + \tfrac{1}{2}P \cdot \delta_2 \tag{90}$$

Now in (d), Fig. 74, let P be gradually applied at 1, producing a deflection Δ_1 at 1, and the work on the beam will be $W_2 = \tfrac{1}{2}P \cdot \delta_2$. Then

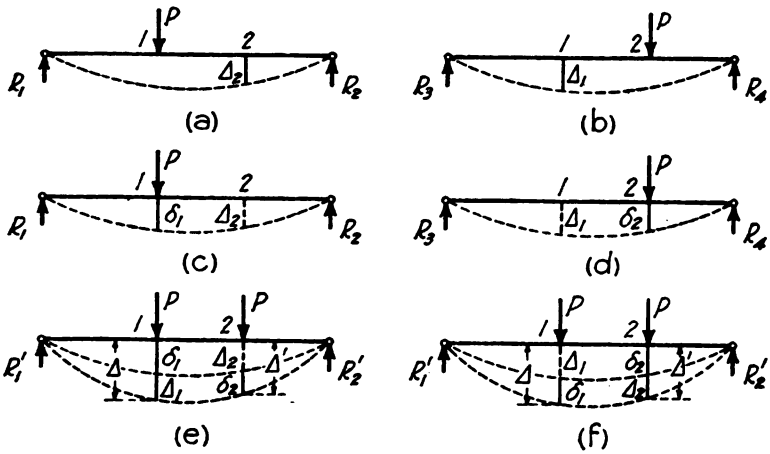

FIG. 74.

if load P be applied at point 1 in (f), Fig. 74, the deflection at point 2 will be Δ_2, and the work due to both loads will be

$$W = \tfrac{1}{2}P \cdot \delta_1 + \tfrac{1}{2}P \cdot \delta_2 + P \cdot \Delta_2 \tag{91}$$

Now the work due to both loads will be independent of the order of the application of the loads, and equating (90) and (91) and solving gives $\Delta_1 = \Delta_2$, which proves the theorem.

Now in (c), Fig. 73, if the deflection due to a load unity at 2 is m at P_1, then the deflection at 2 due to a load unity at P_1 will be m. If load R_2 is applied at 2, the work done in making the elastic curve pass through 2 will be $\tfrac{1}{2}R_2 \cdot c$; while the work of resistance due to a load P_1 will be one half P_1 times the deflection at 2 due to the load P_1, which is equal to $\tfrac{1}{2}P_1 \cdot m$. In like manner the resistance due to P_2 will be $\tfrac{1}{2}P_2 \cdot n$, and

$$R_2 \cdot c = P_1 \cdot m + P_2 \cdot n$$

and

$$R_2 = \frac{P_1 \cdot m + P_2 \cdot n}{c} \tag{88}$$

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To find R_1, take moments about 3 in (a), Fig. 73, and

$$R_1(L_1+L_2)+R_2 \cdot L_2-P_1 \cdot l_1-P_2 \cdot l_2=0$$

and from similar triangles in (c)

$$R_1 \cdot a+R_2 \cdot c-P_1(m+b)-P_2(n-d)=0 \tag{90}$$

Substituting the value of R_2 from (88) in (90), we have

$$R_1 \cdot a=P_1 \cdot b-P_2 \cdot d$$

$$R_1=\frac{P_1 \cdot b-P_2 \cdot d}{a} \tag{87}$$

Since

$$R_1+R_2+R_3=P_1+P_2$$

$$R_3=\frac{-P_1 \cdot e+P_2 \cdot f}{g} \tag{89}$$

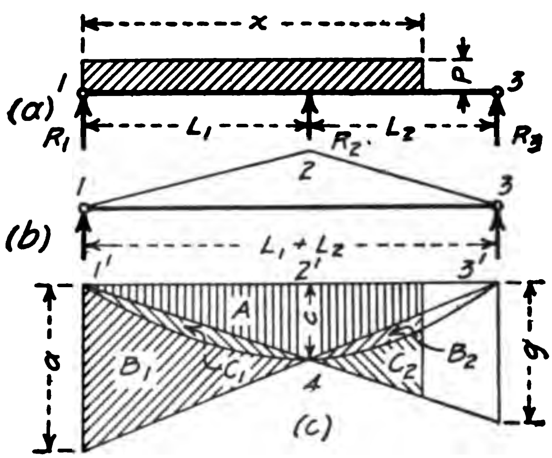

FIG. 74 a.

Uniform Moving Load.—For a uniform load on the beam, the areas of the diagram covered by the uniform load will be used in the place of the ordinates, as in Fig. 73 (see Chapter X, "The Design of

Steel Mill Buildings" for a discussion of influence diagrams). For example, in Fig. 74*a*, the reactions are given by the following formulas:

$$R_1 = \frac{p(\text{area } B_1 - \text{area } B_2)}{a} \tag{92}$$

$$R_2 = \frac{p(\text{area } A + \text{area } B_2 + \text{area } C_1)}{c} \tag{93}$$

$$R_3 = \frac{p(\text{area } C_2 - \text{area } C_1)}{g} \tag{94}$$

For the application of this method to the calculation of the stresses in a continuous girder with four supports, see Chapter XVA, "The Design of Steel Mill Buildings."

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CHAPTER V.

STRESSES IN STATICALLY INDETERMINATE HEAD FRAMES.

Introduction.—Head frames of the 4-post type are statically indeterminate to a greater or less degree depending upon the details of the design. The 4-post head frame in Fig. 75 is statically indeterminate both internally and externally. It is statically indeterminate externally because the reactions cannot be calculated by statics, the distri-

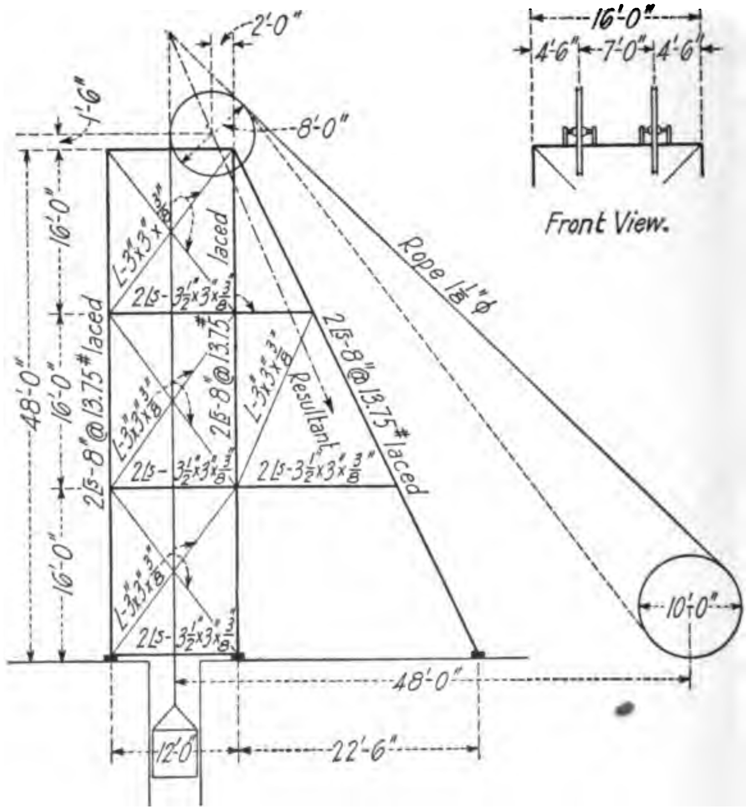

FIG. 75. 4-POST HEAD FRAME.

bution of the load between the tower and back brace not being known; and is statically indeterminate internally because there are redundant members in the 4-post tower and in the bracing between the tower and the back brace.

The dimensions of the head frame and the make-up of the members are given in Fig. 75. The head sheaves are 8 ft. in diameter, the hoisting drum is placed 48 ft. from the center of the shaft. The hoisting rope is 1⅛ in. round with an ultimate strength of 84,000 lbs. The calculations will be made for the right hand or near frame for the full breaking load of one hoisting rope, it being assumed that both ropes break at the same time. Stresses for other conditions may be calculated from the stresses due to 84,000 lbs., the live load stresses being proportional to the loads. The dead load stresses may be calculated in a manner similar to the method given for live loads, or since the dead load stresses are small, they may be calculated with sufficient accuracy by an approximate solution by assuming that the redundant members carry none of the dead load. The wind load stresses may be calculated in the same manner as live load stresses, or as is the general custom the 4-post tower may be assumed to carry all the wind loads, and the bracing calculated upon that assumption.

CALCULATION OF THE STRESSES.—The diagonal bracing in the tower will be assumed as ties not having initial tension, and the redundant diagonals may then be omitted. The back brace and the bracing between the tower and back brace are redundant. There will be two cases, depending upon whether (1) the back brace alone is considered redundant and the bracing is omitted, or (2) where the back brace and the upper horizontal strut are both considered redundant. The solution will cover Case 1, One Redundant Member, and Case 2, Two Redundant Members. With high frames it is often necessary to solve for 3 or even 4 redundant members.

Case 1. One Redundant Member.—The stresses may be calculated by the "Theory of Least Work," or the method used in the calculation of the stresses in a two-hinged arch in Chapter IV may be used.

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Method of Least Work.—The breaking stress of 84,000 lbs. in the hoisting rope produces a resultant which falls inside of the back brace. The resultant of the stress in the rope is replaced by its components, a horizontal force $H = 60{,}000$ lbs. acting in line with the top strut, a vertical force $P_1 = 16{,}000$ lbs. acting in line with the left post, and $P_2 = 124{,}000$ lbs. acting in line with the right vertical post. These external forces were calculated by moments in the usual manner.

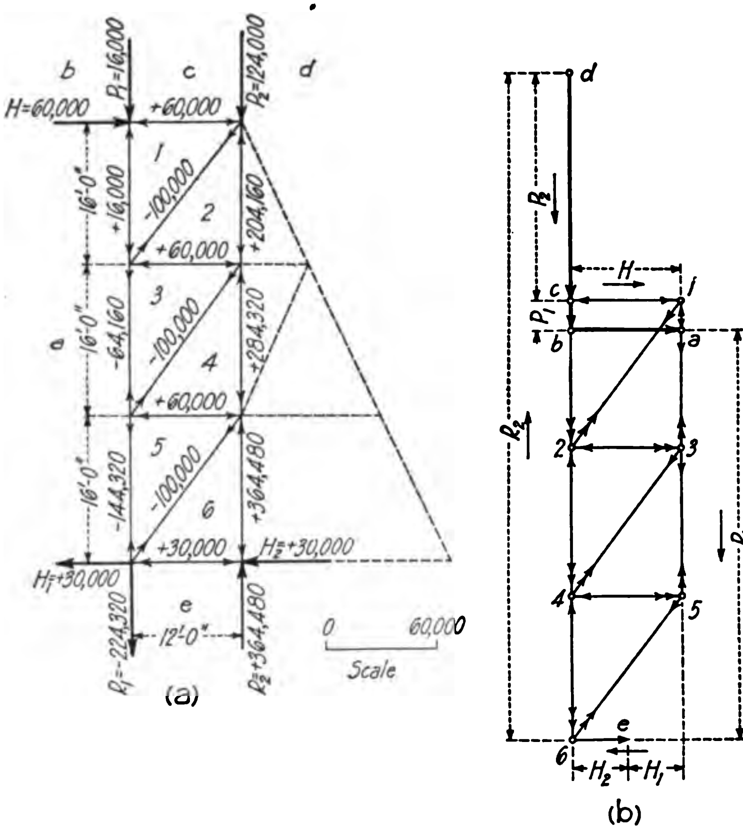

FIG. 76. STRESSES DUE DIRECT LOADING.

The stresses in the tower—the back brace being redundant is omitted—for the external loads H, P_1 and P_2 were calculated in (*b*) by graphic resolution, and the stresses in the members are given in (*a*), Fig. 76.

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The values of $\frac{dS}{dS_{12}} = U$ are calculated in (*b*), Fig. 77, for a load of 1 lb. acting in line of the back brace S_{12}. The values of $\frac{dS}{dS_{12}} = U$ are given in (*a*), Fig. 77.

Work Equations.—The work equations are given in Table XV. The members as marked in Fig. 79, are given in column 1; the cross-sectional areas of the members are given in column 2; the lengths,

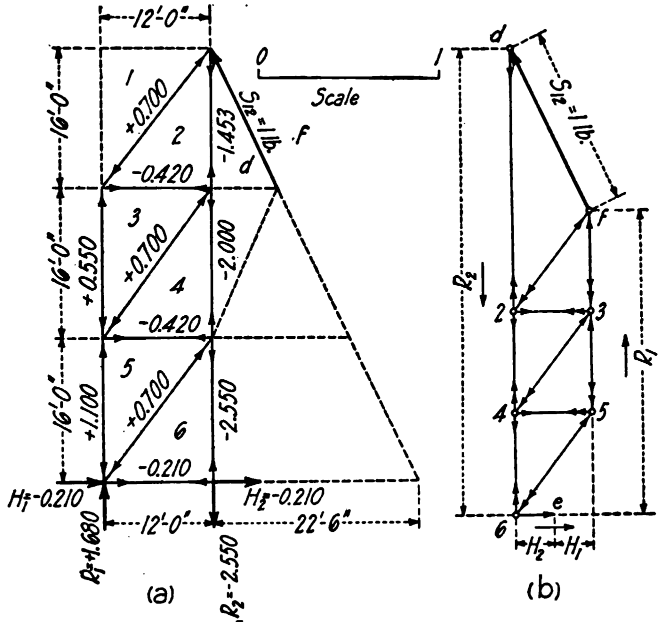

FIG. 77. VALUES OF *U*.

L, of the members are given in column 3; the stresses in the members in terms of S_{12}—the stress in the back brace—the method of calculating which will be given presently are given in column 4; the values of $\frac{dS}{dS_{12}} = U$ are given in column 5; values of $B = \frac{L}{A \cdot E}$, for $E = 30{,}000{,}000$, are given in column 6; while values of $B \cdot S \left(\frac{dS}{dS_{12}}\right)$ are given in

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column 7. The stresses in column 4 may be calculated directly by algebraic resolution, or the stress in each member is equal to the stress due to the direct loads, as given in (*a*), Fig. 76, plus S_{12} times the stress U, in the member due to a unit load. For example the direct stress in S_1 from (*a*), Fig. 76, is 100,000 lbs., while the stress U from (*a*), Fig. 77, is $+0.700$. The true stress in the member S_1 will then be $-100{,}000 + 0.700\,S_{12}$.

The sum of the values of $B \cdot S\left(\frac{dS}{dS_{12}}\right)$ in column 7 for members S_1 to S_{11} is $\frac{dW}{dS_{12}} = \Sigma B \cdot S\left(\frac{dS}{dS_{12}}\right) = -2.4268 + 0.000{,}017{,}177\,S_{12} = y_{12}$. The deformation of the member S_{12} due to a stress of S_{12} will be $0.000{,}002{,}620\,S_{12} = -y_{12}$. Equating the two values of y_{12} and solving, we have $S_{12} = +123{,}000$ lbs.

TABLE XV.

CALCULATION OF WORK EQUATIONS FOR STRESS OF 84,000 LBS. IN HOISTING ROPE.

1	2	3	4	5	6	7
Member.	Area, A, Sq. In.	Length, L, In.	Stress, S, in Terms of S_{12}.	$\frac{dS}{dS_{12}} = U$	$\frac{L}{A \cdot E} = B$	$B \cdot S\left(\frac{dS}{dS_{12}}\right)$
S_1	2.11	240	$-100{,}000+0.700S_{12}$	$+0.700$	0.000,003,800	$-0.2660+0.000{,}001{,}862S_{12}$
S_2	8.08	192	$+204{,}160-1.453S_{12}$	-1.453	0.000,000,792	$-0.2370+0.000{,}001{,}670S_{12}$
S_3	4.60	144	$+60{,}000-0.420S_{12}$	-0.420	0.000,001,044	$-0.0266+0.000{,}000{,}188S_{12}$
S_4	8.08	192	$-64{,}160+0.550S_{12}$	$+0.550$	0.000,000,792	$-0.0280+0.000{,}000{,}238S_{12}$
S_5	2.11	240	$-100{,}000+0.700S_{12}$	$+0.700$	0.000,003,800	$-0.2660+0.000{,}001{,}862S_{12}$
S_6	8.08	192	$+284{,}320-2.000S_{12}$	-2.000	0.000,000,792	$-0.4450+0.000{,}003{,}160S_{12}$
S_7	4.60	144	$+60{,}000-0.420S_{12}$	-0.420	0.000,001,044	$-0.0266+0.000{,}000{,}188S_{12}$
S_8	8.08	192	$-144{,}320+1.100S_{12}$	$+1.100$	0.000,000,792	$-0.1250+0.000{,}000{,}950S_{12}$
S_9	2.11	240	$-100{,}000+0.700S_{12}$	$+0.700$	0.000,003,800	$-0.2660+0.000{,}001{,}862S_{12}$
S_{10}	8.08	192	$+364{,}480-2.550S_{12}$	-2.550	0.000,000,792	$-0.7340+0.000{,}005{,}150S_{12}$
S_{11}	4.60	144	$+30{,}000-0.210S_{12}$	-0.210	0.000,001,044	$-0.0066+0.000{,}000{,}047S_{12}$
						$\frac{dW}{dS_{12}} = \Sigma B \cdot S\left(\frac{dS}{dS_{12}}\right) = y_{12} = -2.4268+0.000{,}017{,}177S_{12}$
S_{12}	8.08	636	S_{12}	$+1.00$	0.000,002,620	$-y_{12}=0.000{,}002{,}620S_{12}$

(1) $-2.4268 + 0.000{,}017{,}177S_{12} = y_{12}$
(2) $0.000{,}002{,}620S_{12} = -y_{12}$
(3) $0.000{,}019{,}797S_{12} = 2.4268$
$S_{12} = +123{,}000$ lbs.

Having the stress $S_{12} = +123{,}000$ lbs. the stresses in the head frame are calculated by graphic resolution as in Fig. 78. The stresses may

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also be calculated by substituting $S_{12} = 123{,}000$ lbs. in the stresses in the members as given in column 4, Table XV. For example the stress in $S_1 = -100{,}000 + 0.700\,S_{12} = -100{,}000 + 0.700 \times 123{,}000 =$

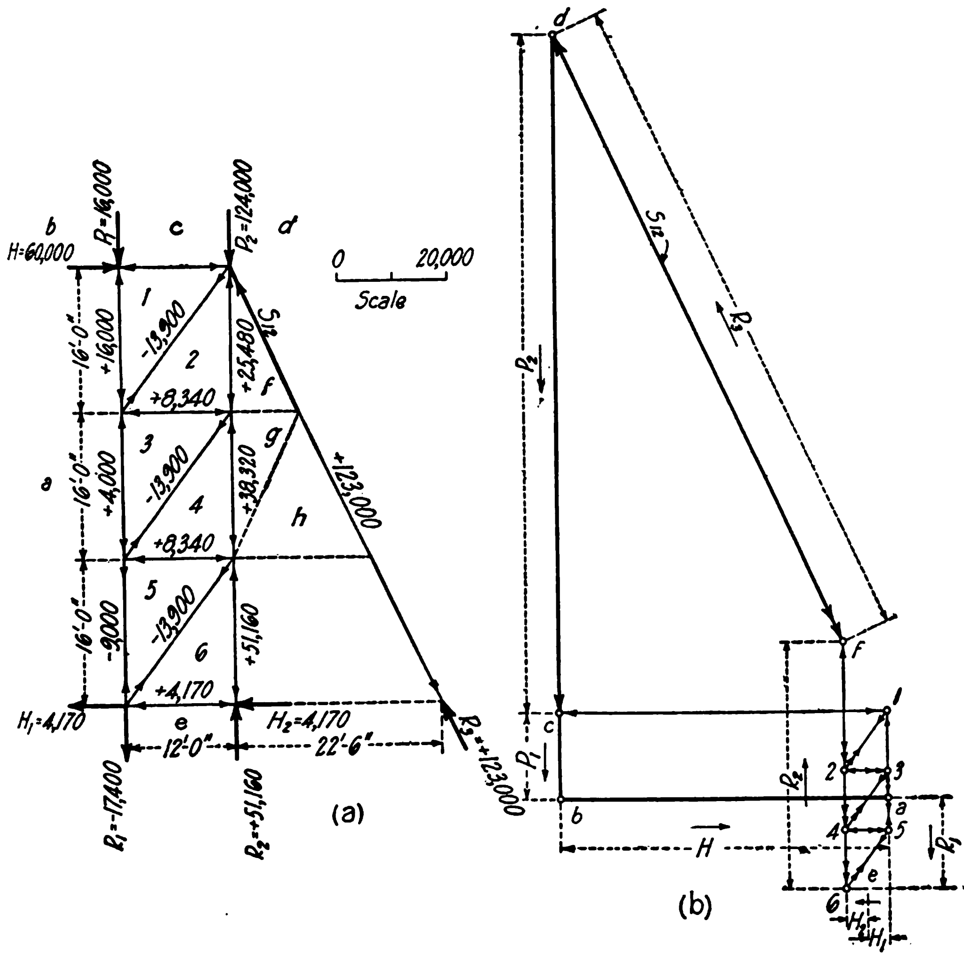

FIG. 78. STRESSES IN HEAD FRAME, BACK BRACE REDUNDANT.

$-100{,}000 + 86{,}100 = -13{,}900$ lbs. The remaining stresses may be calculated in the same manner.

The final stresses in the head frame for live load are given in Fig. 79.

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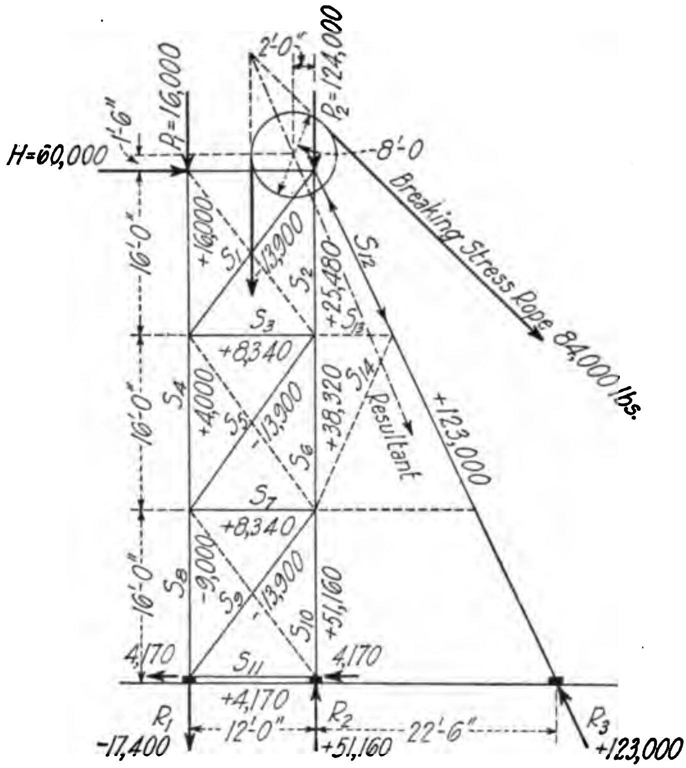

FIG. 79. STRESSES IN HEAD FRAME, BACK BRACE REDUNDANT.

Method of Two-Hinged Arch.—The stresses, S, in the tower due to the external loads, the back brace not acting are calculated as in Fig. 76. The values of U for a load of 1 lb. acting in line with the back brace, are calculated as in Fig. 77. The deformation of the upper right hand point of the head frame in line with the back brace is then calculated in Table XVI. Columns 1, 2 and 3 in Table XVI are the same as the respective columns in Table XV. The stresses S in column 4 are the stresses in the tower when the back brace is not acting, and are obtained from Fig. 76. The deformation of each member is given in column 5; the values of U, obtained from Fig. 77, are given in column 6, while values of $\dfrac{S \cdot U \cdot L}{A \cdot E}$ are given in column 7.

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TABLE XVI.

DEFORMATION AT TOP OF HEAD FRAME DUE TO HOISTING ROPE, BACK BRACE, S_{12} NOT ACTING.

1	2	3	4	5	6	7
Member.	Area, A, Sq. In.	Length, L, In.	Stress, S, Lbs.	$\frac{S \cdot L}{A \cdot E}$	U	$\frac{S \cdot U \cdot L}{A \cdot E}$
S_1	2.11	240	−100,000	−0.380	+0.700	−0.2660
S_2	8.08	192	+204,160	+0.163	−1.453	−0.2370
S_3	4.60	144	+ 60,000	+0.0625	−0.420	−0.0266
S_4	8.08	192	− 64,160	−0.510	+0.550	−0.0280
S_5	2.11	240	−100,000	−0.380	+0.700	−0.2660
S_6	8.08	192	+284,320	+0.225	−2.000	−0.4450
S_7	4.60	144	+ 60,000	+0.0625	−0.420	−0.0266
S_8	8.08	192	−144,320	−0.114	+1.100	−0.1250
S_9	2.11	240	−100,000	−0.380	+0.700	−0.2660
S_{10}	8.08	192	+364,480	+0.287	−2.550	−0.7340
S_{11}	4.60	144	+ 30,000	+0.312	−0.210	−0.0066

$$\Sigma \frac{S \cdot U \cdot L}{A \cdot E} = -2.4268$$

TABLE XVII.

DEFORMATION AT TOP OF HEAD FRAME DUE TO A VALUE OF $S_{12} = 100,000$ LBS.

1	2	3	4	5	6	7
Member.	Area, A, Sq. In.	Length, L, In.	Stress, S', lbs.	$\frac{S' \cdot L}{A \cdot E}$	U	$\frac{S' \cdot U \cdot L}{A \cdot E}$
S_1	2.11	240	+ 70,000	+0.2600	+0.700	0.1862
S_2	8.08	192	−145,300	−0.1150	−1.453	0.1670
S_3	4.60	144	+ 42,000	−0.0430	−0.420	0.0188
S_4	8.08	192	+ 54,800	+0.0434	+0.550	0.0238
S_5	2.11	240	+ 70,000	−0.2600	+0.700	0.1862
S_6	8.08	192	−200,000	−0.1580	−2.000	0.3160
S_7	4.60	144	− 42,000	−0.0430	−0.420	0.0188
S_8	8.08	192	+110,000	+0.0864	+1.100	0.0950
S_9	2.11	240	+ 70,000	+0.2600	+0.700	0.1862
S_{10}	8.08	192	−254,800	−0.2030	−2.550	0.5150
S_{11}	4.60	144	− 20,000	−0.0215	−0 210	0.0047
					$\Sigma \frac{S' \cdot U \cdot L}{A \cdot E}$	= 1.7177
S_{12}	8.08	636	100,000	0.2622	+1.00	0.2620

$$S_{12}\left(\frac{1.7177}{100,000} + \frac{0.2620}{100,000}\right) = 2.4268$$

$$S_{12} = 123,000 \text{ lbs.}$$

The deformation of the upper right hand point of the tower in line with the back brace is $y_{12} = -2.4268$ in. The deformation of the upper

right hand point of the tower for a value of $S_{12}=100,000$ lbs. is calculated in Table XVII, and is $y_{12}=+1.7177$ in. The deformation of S_{12} for a stress of 100,000 lbs. $=0.262$ in. The total deformation will then be

$$S_{12}\left(\frac{1.7177}{100,000}+\frac{0.262}{100,000}\right)=2.4268 \text{ in.}$$

and

$$S_{12}=123,000 \text{ lbs.}$$

Case 2. Two Redundant Members.—If it is assumed that the two upper diagonal braces between the tower and the back brace carry stress

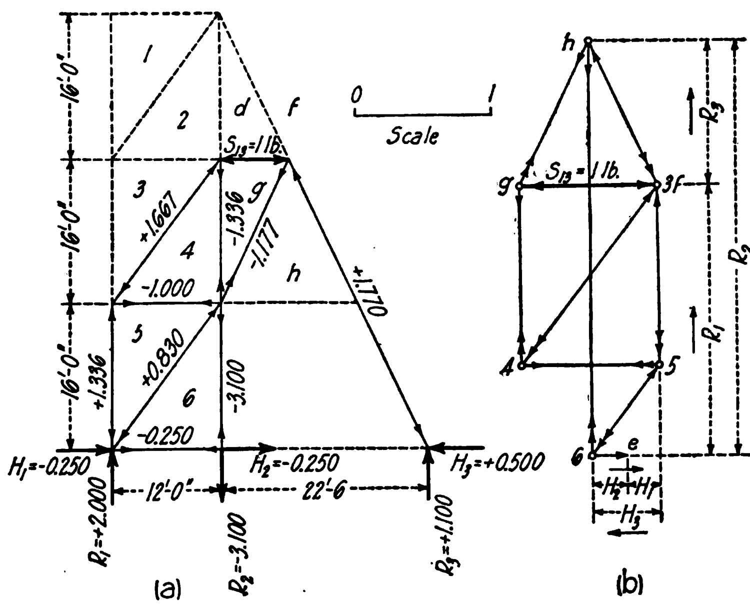

FIG. 80. VALUES OF U'.

the structure has two redundant members. This problem can only be solved by the "Theory of Least Work," the method of the two-hinged arch not being applicable to this problem.

Solution.—The stresses in the tower due to the external loads are calculated in Fig. 76, and are the same as in Case 1. The stresses, U,

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in the members due to a stress of 1 lb. in the back brace are calculated in Fig. 77. The values of U are the values of $\frac{dS}{dS_{12}}$ for the corresponding members. The stresses, U', in the members due to a stress of 1 lb. in the top horizontal strut are calculated in Fig. 80. The values of U' are the values of $\frac{dS}{dS_{13}}$ for the corresponding members.

Work Equations.—The work equations are given in Table XVIII. The values in columns 1, 2 and 3 are the same as in Table XV. The stresses in the members in terms of the unknown stresses S_{12} and S_{13} are given in column 4—these stresses may be calculated directly by algebraic resolution, or may be found by adding to the stresses in the tower due to direct loads the unknown stress S_{12} multiplied by the values of $\frac{dS}{dS_{12}}$ (U), and the unknown stress S_{13} multiplied by $\frac{dS}{dS_{13}}$ (U'). The values of $\frac{dS}{dS_{12}}=U$ were calculated in Fig. 77; while the values of $\frac{dS}{dS_{13}}=U'$ were calculated in Fig. 80. The values of B in column 7 were calculated for $E=30{,}000{,}000$. The values of $B\cdot S\left(\frac{dS}{dS_{12}}\right)$ are given in column 8, and of $B\cdot S\left(\frac{dS}{dS_{13}}\right)$ are given in column 9. Now the deformation of the back brace will be equal to the deformation of the tower in line of the back brace, and from column 8 the deformations are

$$-y_{12}=0.000{,}000{,}874\,S_{12} \qquad (a)$$

and

$$y_{12}=0.000{,}018{,}925\,S_{12}+0.000{,}018{,}646\,S_{13}-2.4268 \qquad (b)$$

Adding equations (a) and (b) gives

$$0.000{,}019{,}799\,S_{12}+0.000{,}018{,}646\,S_{13}=2.4268 \qquad (c)$$

From column 9 the deformation of the strut, and the tower in line with

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the strut are $$-y_{13} = +0.000{,}000{,}650\,S_{13} \qquad (d)$$

and $$y_{13} = 0.000{,}018{,}646\,S_{12} + 0.000{,}031{,}999\,S_{13} - 2.3755 \qquad (e)$$

Adding equations (d) and (e) gives

$$0.000{,}018{,}646\,S_{12} + 0.000{,}032{,}649\,S_{13} = 2.3755 \qquad (f)$$

Solving equations (c) and (e) for S_{12} and S_{13} gives

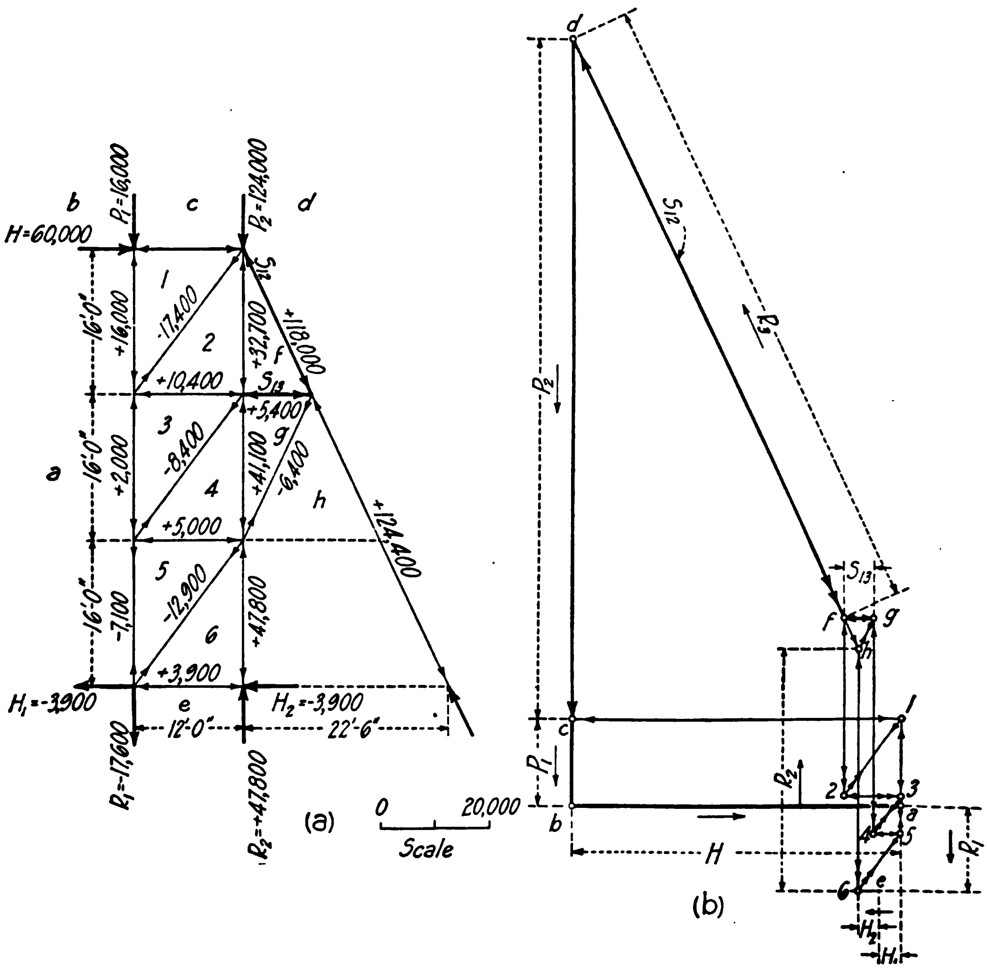

FIG. 81. STRESSES IN HEAD FRAME, TWO MEMBERS REDUNDANT.

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TABLE XVIII.

CALCULATIONS OF WORK EQUATIONS FOR STRESS OF 84,000 LBS. IN ROPE.

1	2	3	4	5	6	7	8	9
Member.	Area, A, Sq. In.	Length, L, In.	Stresses in Terms of S_{12} and S_{13}.	$\frac{dS}{dS_{12}}=U$	$\frac{dS}{dS_{13}}=U'$	$\frac{L}{A\cdot E}=B$	$B\cdot S\left(\frac{dS}{dS_{12}}\right)$	$B\cdot S\left(\frac{dS}{dS_{13}}\right)$
S_1	2.11	240	$-100,000+0.700S_{12}$	$+0.700$		0.000,003,800	$-0.2660+0.000,001,862S_{12}$	
S_2	8.08	192	$+204,160-1.453S_{12}$	-1.453		0.000,000,792	$-0.2370+0.000,001,670S_{12}$	
S_3	4.60	144	$+60,000-0.420S_{12}$	-0.420		0.000,001,044	$-0.0266+0.000,000,188S_{12}$	
S_4	8.08	192	$-64,160+0.550S_{12}$	$+0.550$		0.000,000,792	$-0.0280+0.000,000,238S_{12}$	
S_5	2.11	240	$-100,000+0.700S_{12}$ $+1.667S_{13}$	$+0.700$	$+1.667$	0.000,003,800	$-0.2660+0.000,001,862S_{12}$ $+0.000,004,320S_{13}$	$-0.6350+0.000,004,320S_{12}$ $+0.000,010,600S_{13}$
S_6	8.08	192	$+284,320-2.000S_{12}$ $-1.336S_{13}$	-2.000	-1.336	0.000,000,792	$-0.4450+0.000,003,160S_{12}$ $+0.000,002,120S_{13}$	$-0.3020+0.000,002,120S_{12}$ $+0.000,001,420S_{13}$
S_7	4.60	144	$+60,000-0,420S_{12}$ $-S_{13}$	-0.420	-1.000	0.000,001,044	$-0.0266+0.000,000,188S_{12}$ $+0.000,000,440S_{13}$	$-0.0625+0.000,000,440S_{12}$ $+0.000,001,044S_{13}$
S_8	8.08	192	$-144,320+1.100S_{12}$ $+1.336S_{13}$	$+1.100$	$+1.336$	0.000,000,792	$-0.1250+0.000,000,950S_{12}$ $+0.000,001,160S_{13}$	$-0.1530+0.000,001,160S_{12}$ $+0.000,001,420S_{13}$
S_9	2.11	240	$-100,000+0.700S_{12}$ $+0.830S_{13}$	$+0.700$	$+0.830$	0.000,003,800	$-0.2660+0.000,001,862S_{12}$ $+0.000,002,260S_{13}$	$-0.3150+0.000,002,260S_{12}$ $+0.000,002,620S_{13}$
S_{10}	8.08	192	$+364,480-2.550S_{12}$ $-3.100S_{13}$	-2.550	-3.100	0.000,000,792	$-0.7340+0.000,005,150S_{12}$ $+0.000,006,250S_{13}$	$-0.9000+0.000,006,250S_{12}$ $+0.000,007,600S_{13}$
S_{11}	4.60	144	$+30,000-0.210S_{12}$ $-0.250S_{13}$	-0.210	-0.250	0.000,001,044	$-0.0066+0.000,000,047S_{12}$ $+0.000,000,056S_{13}$	$-0.0080+0.000,000,056S_{12}$ $+0.000,000,065S_{13}$
S_{14}	2.11	212	$-1.1770S_{13}$		-1.1770	0.000,003,460		$+0.000,004,800S_{13}$
S_{15}	8.08	424	$S_{12}+1.1770S_{13}$	1.000	$+1.1770$	0.000,001,748	$0.000,001,748S_{12}$ $+0.000,002,040S_{13}$	$+0.000,002,040S_{12}$ $+0.000,002,420S_{13}$

$$\Sigma B\cdot S\left(\frac{dS}{dS_{12}}\right)=-2.4268+0.000,018,925S_{12}+0.000,018,646S_{13}\qquad \Sigma B\cdot S\left(\frac{dS}{dS_{13}}\right)=-2.3755+0.000,018,646S_{12}+0.000,031,999S_{13}$$

S_{12}	8.08	212	S_{12}	1.000		0.000,000,874	$0.000,000,874S_{12}$	
S_{13}	4.60	90	S_{13}		1.000	0.000,000,650		$0.000,000,650S_{13}$

$$y_{12}=0.000,018,925S_{12}+0.000,018,646S_{13}-2.4268 \qquad y_{13}=0.000,018,646S_{12}+0.000,031,999S_{13}-2.3755$$
$$-y_{12}=0.000,000,874S_{12} \qquad -y_{13}= \qquad +0.000,000,650S_{13}$$

$$(1)\quad 0.000,019,799S_{12}+0.000,018,646S_{13}=2.4268 \qquad (2)\quad 0.000,018,646S_{12}+0.000,032,649S_{13}=2.3755$$
$$(3)\quad 0.000,010,650S_{12}+0.000,018,646S_{13}=1.3550$$
$$0.000,009,149S_{12}=1.0708$$

$S_{12}=+118,000$ lbs.

Substituting in (1), $S_{13}=+\ 5,400$ lbs.

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$$S_{12} = +\ 118{,}000 \text{ lbs.}$$
$$S_{13} = +\ \ \ 5{,}400 \text{ lbs.}$$

The stresses in the members of the head frame were calculated by graphic resolution in Fig. 81. The stresses in members may be calculated by substituting the true values of S_{12} and S_{13} in column 4, Table XVIII. For example the stress in

$$S_5 = -\ 100{,}000 + 0.700\,S_{12} + 1.667\,S_{13}$$
$$S_5 = -\ 100{,}000 + 82{,}600 + 9{,}000$$
$$= -\,8{,}400 \text{ lbs.}$$

The stresses in the different members of the head frame are given in Fig. 82.

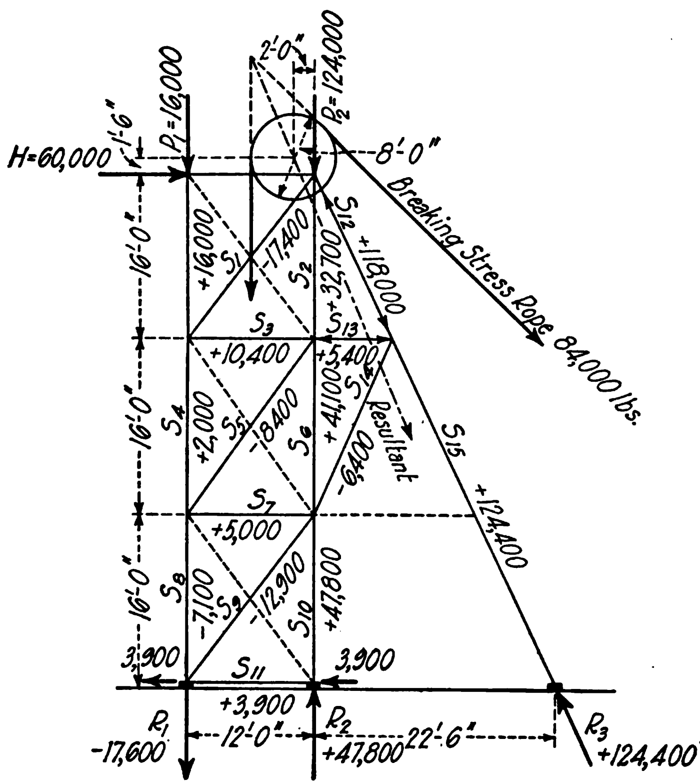

FIG. 82. STRESSES IN HEAD FRAME, TWO MEMBERS REDUNDANT.

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CHAPTER VI.

The Design of Head Frames.

Introduction.—The selection of the type of head frame will be governed principally by the number of compartments in the shaft. Where the compartments of the shaft are in line and the sheaves can be placed parallel, the A-type of head frame has the advantage of simplicity and economy. Where it is necessary to hoist from more than two compartments, or where the head sheaves must be placed in tandem, it becomes necessary to use a head frame of the 4-post type. Where there are several compartments it may be necessary to use six or more main posts for the tower with three or more back brace columns, as in the head frame for the Phillips coal tipple, Fig. 143. With the A-type of head frame the head sheaves are commonly placed with their center lines parallel with the center line of the back braces and the guides are placed in front of the main columns. This arrangement makes it possible to fully brace the head frame and gives a simple structure in which the stresses are statically determinate. In a head frame of the 4-post type the main posts are placed around the shaft, and it is necessary to remove part of the bracing in order to dump the ore. This makes it difficult to brace the main structure and further complicates the calculation of the stresses in a structure which is already statically indeterminate.

In the A-type of head frame the main columns are placed in a vertical plane parallel to the guides, and have a batter of 1 to 2 in. to the foot in this plane. The main back brace is usually placed at an angle of approximately 30 degrees with the vertical. Head frames of the 4-post type are commonly built with main posts in vertical planes parallel to the guides and are battered in this plane 1 to 1½ in. to the foot. The Elkton steel head frame, Fig. 99, has the main tower columns vertical and has side braces to give lateral stability. For a light head frame this is a satisfactory solution as it gives a rigid and economical frame.

In the design of heavy head frames the main columns are commonly made of two channels laced or of two channels with a cover plate. The main columns of the head frame of the Phillips coal tipple shown in Fig. 143 are composed of four Z-bars and one plate; the main columns of the steel head frames for the Diamond mine and the High

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Ore mine shown in Fig. 87, are made of two built-up channels with a heavy cover plate; while the main posts of the Elkton steel head frame, Fig. 99, are made of single 6 in. $\times$ 6 in. angles. Bethlehem H-beams have also been used for columns and posts. The bracing of heavy steel head frames is commonly made of two channels laced or battened, while the bracing of light steel head frames is made of angles.

The hoisting engine is usually placed so that the angle of the hoisting rope to the horizontal will be from 40 to 45 degrees. Under these conditions the resultant of the stresses in the hoisting rope will fall inside the head frame, and there will be no uplift on the main columns. In the Lake Superior copper country the hoist is usually placed at a considerable distance from the rock house, so that the resultant of the stress in the rope strikes in front of the main brace; in which case it is necessary to provide for the uplift on the main columns. It is desirable in coal tipples to have the hoist at a considerable distance from the tipple on account of the danger from fire, and the uplift on head frames for coal tipples should be carefully investigated.

ALLOWABLE STRESSES.—Head frames should be designed for (1) dead load; (2) a working load; (3) a breaking load in one or both ropes, and (4) for wind loads.

1. **Dead Loads.**—The dead load stresses are due to the weight of the head frame, the head sheaves, and concentrated loads such as ore-bins, etc. The allowable stresses for dead loads are the same as for the design of steel frame buildings given in Appendix I (factor of safety of 4) which are as follows in lbs. per sq. in.

Tension	16,000
Compression	$16{,}000-70\frac{l}{r}$

where l = length of the member in inches, and r = the least radius of gyration in inches.

Rivets and pins, bearing	22,000
Rivets and pins, shear	11,000
Pins, bending on extreme fiber	24,000
Plate girder webs, shear on net section	10,000
Bearing on granite masonry and 1:2:4 Portland cement concrete	500
Bearing on sandstone and limestone masonry	400
Expansion rollers, per lineal inch	600 D

where D = diameter of roller in inches.

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2. **Working Load.**—The working load will be

$$K = 2W + R + F \tag{1}$$

where K = stress in rope in lbs. at the sheaves at instant of picking up the load;
W = gross load in lbs.;
R = weight of rope in lbs.;
F = friction in lbs. = $(W + R)f$, where
f = coefficient of friction, equal to about 0.01 to 0.02 for vertical shafts.

The working loads should be considered live loads, and the stresses should be treated in the same manner as the live load stresses in bridges, with allowable stresses one half the allowable stresses for dead loads or for steel frame buildings as given in Appendix I (factor of safety 8).

3. **Breaking Load.**—The maximum stresses in a head frame will occur when a cage or skip sticks in the shaft, or when the brakes on the hoisting engine are suddenly applied and the hoisting rope is broken. Where the hoist may be run unbalanced it is possible for both ropes to be broken at the same time. The maximum stresses in the transverse frames will occur when one rope breaks, while the maximum stresses in the back brace and front posts will occur when both ropes break. The head frame should be designed so that no stress will exceed the elastic limit of the steel when one or both hoisting ropes break. The allowable stresses for breaking the hoisting ropes may be taken as equal to one and one half times the allowable stresses for dead load, or for steel frame buildings as given in Appendix I (factor of safety of $2\frac{2}{3}$).

4. **Wind Loads.**—The wind load stresses when combined with the dead load stresses and twice the working load stresses, should not cause a stress greater than 20,000 lbs. per sq. in. Where the wind stresses are less than 25 per cent of the sum of the dead load stresses and twice the working load stresses, no additional material will be required in the sections, but where the wind stress in any member exceeds the sum of the dead load stress and twice the live load stress by more than 25 per cent, the area of the member should be increased, so that the total unit stress is not more than 25 per cent greater than the actual dead load stress plus twice the actual live load stress in the member. Wind load stresses need not be combined with breaking load stresses.

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Thickness of Material.—Mine shafts at which head frames are placed are usually up-casts, so that the steel head frame is subjected to the action of a moist laden current of air which rapidly destroys ordinary paints and causes rapid corrosion. To provide for corrosion the minimum thickness of structural steel sections and plates should be $\frac{5}{16}$ in., and the structure should be painted at frequent intervals with a structural paint that will resist the action of corrosive gases. The best paints for painting head frames and coal tipples are graphite or carbon paints. Goheen's carbonizing coating has also given good results. A good red lead paint applied carefully and at frequent intervals will give good results under average conditions. In painting structural steel it is very important that the materials be pure and be carefully mixed, that the surface of the steel be clean, dry and warm, that the paint be carefully brushed in, and that a sufficient number of coats be applied to give the necessary protection. It is best to have one coat of paint applied at the shop, the abraded spots and rivet heads being touched up before erection, and two coats of paint applied after the erection is completed. The three coats of paint should preferably be of different shades or colors to make sure that all parts be covered. Head frames and coal tipples should be repainted every 3 to 4 years.

DESIGN OF A STEEL HEAD FRAME.—The sections of the members of the A-type steel head frame in which the stresses were found in Chapter III will now be calculated.

Front Posts.—The length of a section of the posts will be $l = 12$ ft. 6 in. $= 150$ in.

The stresses (Table IX) are as follows:

Dead load stress	$= +\ 13{,}800$ lbs.
Working load stress	$= +\ 6{,}900$ lbs.
Breaking load stress	$= +\ 43{,}700$ lbs.
Wind load compression	$= +\ 17{,}600$ lbs.
Wind load tension	$= -\ 9{,}700$ lbs.

Before the allowable unit stress can be determined the make-up of the section must be known. A section composed of two 7″ channels @ 12.25 lbs., area of both channels $= 7.20$ sq. in. (Carnegie, p. 101, or Cambria, p. 164), with flanges turned out and laced on both sides, will be assumed, and its efficiency will be investigated. The flanges are 2.198 in. wide, and the channels will be placed 7.0 in. back to back. The moments of inertia about both rectangular axes will be equal if the

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channels are 3.99 in. apart (Carnegie, p. 102), so that the least radius of gyration will be about the axis normal to the webs of the channels, $=2.59$ in. The allowable unit stress in compression is $P=16{,}000 - 70\frac{150}{2.59} = 11{,}940$ lbs. The ratio of l/r must not be greater than 100 (Specifications, § 231). The area required for dead loads will then be $= 13{,}800 \div 11{,}940 = 1.16$ sq. in.; the area for working load $= 6{,}900 \div \frac{1}{2}(11{,}940) = 1.16$ sq. in. The total area required for dead loads and working load $= 1.16 + 1.16 = 2.32$ sq. in.

The direct wind load compression is more than 25 per cent of the dead load plus twice the working load, and must be considered. The area required to take dead load stress, working load stress, and wind stress $= (13{,}800 + 2 \times 6{,}900 + 17{,}600) \div (11{,}940 \times 1.25) = 3.02$ sq. in.

The negative wind stress does not produce a reversal with the dead load stress, and need not be considered.

The area required to take the breaking load $= 43{,}700 \div (\frac{3}{2} \times 11{,}940) = 2.44$ sq. in. The area required to take dead loads and breaking load $= 1.16 + 2.44 = 3.60$ sq. in.

The section has an area of 7.20 sq. in., which appears extravagant. Two 7″ channels @ 9.75 lbs. with an area of 5.7 sq. in. will give sufficient area but the metal is only 0.21 in. thick which is less than $\frac{5}{16}$ in., the minimum thickness for main members (Specifications, § 235). Two 6″ channels @ 10.5 lbs. will give the required area, but will not detail as well with the back braces as the section selected.

Back Braces.—The length of one section of the back brace is $l = 14$ ft. 7 in $= 175$ in.

The stresses are as follows:

Dead load stress	$= +\ 3{,}500$ lbs.
Working load stress	$= +\ 18{,}500$ lbs.
Breaking load stress	$= +\ 116{,}000$ lbs.
Wind load compression	$= +\ 13{,}300$ lbs.
Wind load tension	$= -\ 15{,}200$ lbs.

The section will be assumed as made of two 8″ channels @ 13.75 lbs., area of both channels 8.08 sq. in., with flanges turned out. The backs of the channels will be taken 7 in. apart the same as in the front posts. With a spacing of 4.72 in. the moments of inertia about both rectangular axes will be equal (Carnegie, p. 102). The least radius

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of gyration of the section will then be $r = 2.98$ in., the radius of gyration of the channels about an axis at right angles to the web. The allowable unit stress for compression is then $P = 16{,}000 - 70\,\frac{175}{2.98} =$ 11,875 lbs. The ratio of l/r must not be greater than 100 (Specifications, § 231). The area required for dead loads will be $= 3{,}500 \div 11{,}875 = 0.30$ sq. in.; the area for working load $= 18{,}500 \div \frac{1}{2}\,(11{,}875) = 3.20$ sq. in. The total area required for dead loads and working load $= 0.30 + 3.20 = 3.50$ sq. in.

The direct wind compression is more than 25 per cent of the dead load plus twice the working load and must be considered. The area required to take the dead load stress, the working load stress, and the wind load compression $= (3{,}500 + 2 \times 18{,}500 + 13{,}300) \div (11{,}875 \times 1.25) = 3.70$ sq. in.

The wind load tension is $-15{,}200$ lbs., giving a tension of $-15{,}200 + 3{,}500 = -11{,}700$ lbs., when the dead load only is acting. The connections must be designed for tension as well as compression, but no increase in section is necessary, as the unit stress is small.

The area required to take the breaking load $= 116{,}000 \div (\frac{3}{2} \times 11{,}875) = 6.52$ sq. in. The area required to take the dead load stress and breaking load $= 0.30 + 6.52 = 6.82$ sq. in.

A recalculation shows that two 7″ channels @ 12.25 lbs. are sufficient, but the section first assumed will be used.

Strut D–d (5–6).—The strut has a length $l = 22$ ft. 6 in. $= 270$ in. The stresses are

Dead load stress	$= -1{,}300$ lbs.
Wind load compression	$= +6{,}800$ lbs.
Wind load tension	$= -3{,}300$ lbs.

The wind load stress produces a reversal of $-1{,}300 + 6{,}800$ lbs. $= + 5{,}500$ lbs., making it necessary to design the member both as a tie and as a strut. The specifications (§ 231) require that secondary compression members shall not have a length greater than 140 times the least radius of gyration of the member, from which the least value of the radius of gyration will be $= 270 \div 140 = 1.93$ in. Two channels will be used for the strut with the webs vertical and 7 in. between backs and with lacing on top and bottom, the same as the front posts and the back braces. The radius of gyration of a 5″ channel @ 9 lbs. is 1.83 in., while the radius of gyration of a 6″ channel @ 10.5 lbs. is

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2.21 in. There will be some vibration in the member and it will be made of two 6″ channels @ 10.50 lbs., area = 6.18 sq. in., laced top and bottom.

The allowable unit stress in compression for dead loads is $P = 16{,}000 - 70\frac{270}{2.21} = +7{,}400$ lbs. The area required to take a dead load tension of $-1{,}300$ lbs. and a wind load compression of $+6{,}800$ lbs. $= 5{,}500 \div 7{,}400 \times 1.25 = 0.60$ sq. in.

The area required to take a dead load tension of $-1{,}300$ lbs. and a wind load tension of $-3{,}300$ lbs. is $= (-1{,}300 - 3{,}300) \div (16{,}000 \times 1.25) = 0.23$ sq. in. The net area of the section will be found (see Table XLVI, Chapter XIV), by deducting the area of 4 rivet holes from the area of the member $= 6.18 - 4 \times (\frac{3}{4} + \frac{1}{8}) \times 0.318 = 5.06$ sq. in.

Struts 1–2 and 3–4 will be made of two 6″ channels @ 10.5 lbs. the same as 5–6.

Strut $D_1 - D_2$ (13–14).—The strut has a length $l = 20$ ft. 3 in. $= 243$ in.

The stresses are

$$\begin{aligned} &\text{Breaking load stress} = +\ \ 1{,}100 \text{ lbs.} \\ &\text{Wind load stress} \quad\ \ = +12{,}800 \text{ lbs.} \end{aligned}$$

The section will be assumed as two 6″ channels @ 10.5 lbs., the same as strut 5–6.

The allowable unit stress in compression for dead loads is

$$P = 16{,}000 - 70\frac{243}{2.21} = +8{,}300 \text{ lbs.}$$

The section is safe for both breaking loads and wind stress. Struts 11–12 and 9–10 will have the same section as strut 13–14.

Strut $d_1 - d_2$ (21–22).—The strut has a length $l = 20$ ft. 3 in. $=$ 243 in.

The stresses are

$$\begin{aligned} &\text{Breaking load stress} = +2{,}500 \text{ lbs.} \\ &\text{Wind load stress} \quad\ \ = +8{,}100 \text{ lbs.} \end{aligned}$$

Two 6″ channels @ 10.50 lbs., the same as for strut 13–14, will be used. Struts 19–20 and 17–18 will have the same sections as strut 21–22.

Bracing E–d (6–7).—The length is $l = 25$ ft. 9 in. $= 309$ in.

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The stresses are

Dead load stress	$= +\ 2{,}200$ lbs.
Wind load compression	$= +\ 5{,}300$ lbs.
Wind load tension	$= -\ 5{,}800$ lbs.

It will be necessary to design the member both as a tie and a strut. The least allowable radius of gyration (Specifications, § 231) is $r = 309 \div 140 = 2.19$. The least radius of gyration of a 6″ channel @ 10.5 lbs., is $r = 2.21$, so that the member will be made of two 6″ channels @ 10.50 lbs., with webs vertical and backs of webs 7 in. apart. The member is safe for both tension and compression. The same sections will be used for the remainder of the bracing.

Sheave Girders.—The head sheaves will be carried on built plate girder diaphragms which are carried on sheave girders in the plane of the front posts and back braces. The bracing, Fig. 44, will be dropped at the upper ends so that the sheave girders can be put in place. To make an accurate calculation of the sheave girders it will be necessary to have a detail drawing giving the spacing of the diaphragms. An approximate but safe solution will be made, using the data in Fig. 46.

Sheave Girder on Back Braces.—The values of P_1 and P_2 will be equal to the values in Fig. 46 multiplied by $R_2 \div E$ in Fig. 47 = values in Fig. 46 multiplied by 0.75. The values of P_1 and P_2 in Fig. 46, the weight of the framework being omitted, will then be

Case 1, working load,	$P_1 = 18{,}450$,	$P_2 = 18{,}450$
Case 2, breaking one rope,	$P_1 = 5{,}850$,	$P_2 = 116{,}400$
Case 3, breaking two ropes,	$P_1 = 116{,}400$,	$P_2 = 116{,}400$

For Case 1 the bending moment will be $M = 18{,}450 \times 42 = 814{,}900$ in.-lbs.

For Case 3 the bending moment will be $M_1 = 116{,}400 \times 42 = 4{,}878{,}000$ in.-lbs.

Try two 18″ channels @ 60 lbs. From p. 172, Cambria, $I = 703.3$ in.^{4} With a fiber stress of 16,000 lbs. per sq. in. the allowable bending moment for two 18″ channels @ 60 lbs. will be

$$M = \frac{S \cdot I}{c} = \frac{16{,}000 \times 2 \times 703.3}{9} = 2{,}500{,}000 \text{ in.-lbs.}$$

which is safe for working loads but when multiplied by 3/2 is not sufficient for the breaking loads.

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Assume that each sheave girder is made of two plate girders, each girder having a web plate $30'' \times \frac{3}{8}''$ and flanges made of 2 angles $5'' \times 3'' \times \frac{5}{16}''$ placed back to back with long legs out. From Cambria, p. 182, or Carnegie, p. 111, the area of 2 angles $5'' \times 3'' \times \frac{5}{16}'' = 4.80$ sq. in., and the distance of the neutral axis from the backs of the angles is $x = 0.68$ in. The effective depth of the girder is then $d = 30 - 2 \times 0.68 = 28.64$ in. With $\frac{3}{4}$ in. rivets the net area after deducting one rivet hole from each leg, from Table XLVI, is $4.80 - 2 \times 0.27 = 4.26$ sq. in.

From equation (129), Chapter XIV, the resisting moment of two girders is

$$M = 16{,}000 \times 2 \times 4.26 \times 28.64$$
$$= 3{,}922{,}000 \text{ in.-lbs.}$$

With an allowable unit stress of 24,000 lbs. $M_1 = 5{,}883{,}000$ in.-lbs., which is safe for breaking loads. Net area of web deducting six rivet holes $= 11.25 - 6 \times 0.33 = 9.27$ sq. in. The actual shear for breaking loads is $S = 116{,}400 \div 9.27 = 12{,}550$ lbs. per sq. in. The allowable shear on net section of the web for breaking loads is $S = \frac{3}{2} \times 10{,}000 = 15{,}000$ lbs. per sq. in.

Stiffeners should be provided at the points of concentrated loading and at the ends of the girder.

Sheave Girder on Front Posts.—The values of P_1 and P_2 will be equal to the values in Fig. 46 multiplied by $R_1 \div E$ in Fig. 47 = values in Fig. 46 multiplied by 0.28.

The values of P_1 and P_2 in Fig. 46, the weight of the framework being omitted, will then be

Case 1, working load,	$P_1 = 6{,}890$,	$P_2 = 6{,}890$
Case 2, breaking load, one rope	$P_1 = 2{,}190$,	$P_2 = 43{,}460$
Case 3, breaking load, two ropes,	$P_1 = 43{,}460$,	$P_2 = 43{,}460$

For Case 1 the bending moment will be $M = 6{,}890 \times 42 = 287{,}380$ in.-lbs.

For Case 3 the bending moment will be $M_1 = 43{,}460 \times 42 = 1{,}825{,}320$ in.-lbs.

Try two 15″ channels @ 33 lbs. From Cambria, p. 170, or Carnegie, p. 101, $I = 312.6$ in.^{4}.

With a fiber stress of 16,000 lbs. per sq. in. the allowable bending moment for two 15″ channels @ 33 lbs. will be

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TABLE XIX.

DATA ON STEEL HEAD FRAMES.

	Description.	Depth of Mine, Ft.	Height of Frame, Ft. In.	Diameter of Sheaves, Ft. In.	Size of Hoisting Rope, In	Method of Hoisting.	Weight of		Weight of Ore, Lbs.	Rate of Hoisting.		Weight of Head Frame, Lbs.
							Skip, Lbs.	Cage, Lbs.		Ft. per Min.	Tons per Day.	
1	Sibley Mine, Ely, Minn.	726 (designed for 2,000)	140-0	12-0	$1\frac{3}{8}$	Skips	5,000	3,500	14,000	2,000		576,663
2	High Ore, Butte, Mont.	2,800	100-0	10-0	$7\times\frac{1}{2}$	Skips	7,000		14,000	1,000	1,200	292,000
3	Diamond, Butte, Mont.	2,800	100-0	10-0	$7\times\frac{1}{2}$	Skips	7,000		14,000	1,000	1,200	318,000
4	New Leonard, Butte, Mont.	1,679 (designed for 3,500)	140-0	12-0	$1\frac{1}{2}$	Skips	7,500		10,000	2,800		346,425
5	Inland Steel Co., Hibbing, Minn	225	76-0	6-0	$1\frac{1}{8}$	Skips	3,700		6,700			79,000
6	Elkton, Elkton, Colo.		55-0	5-0	$3\frac{1}{2}\times\frac{3}{8}$				15,200 working load			
7	Cia. Minera de Penoles, Bermejillo, Mex.	1,000	90-0	7-0	$1\frac{1}{8}$	Skips	5,000		10,000			80,000
8	Tonopah-Belmont, Tonopah, Nev.	1,420	75-0	7-0	1					1,000		63,000
9	Copper Queen, Bisbee, Ariz.	1,700	60-0	7-0	$1\frac{1}{4}$	Skips	5,990		3,700		2,000	35,250
10	Union Shaft, Virginia, Nev.	2,000	50-0	7-0	1	Cages		1,200	2,400	1,000	500	42,000
11	Speculator, Butte, Mont.		50-0	7-0	$7\times\frac{1}{2}$	Skips						42,200
12	Basin & Bay State, Basin, Mont.		70-0	10-0	$1\frac{1}{4}$	Skips						79,000
13	Steward, Butte, Mont.		55-0	7-0	$7\times\frac{1}{2}$	Skips			10,000			45,000
14	Anaconda, Butte, Mont.	2,400	58-8	10 0	$7\times\frac{1}{2}$	Skips	7,000		14,000	1,000	1,200	74,700
15	Quincy Rock House, No. 2, Hancock, Mich.	6,000 (inclined 57°)	119-3	12-0	$1\frac{1}{2}$	Skips	10,000		168 cu. ft.		2,400	839,000
16	St. Lawrence, Butte, Mont.	2,100	97-0	10-0	$7\times\frac{1}{2}$	Skips	7,000		14,000	1,000	1,200	117,000

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$$M = \frac{S \cdot I}{c} = \frac{16{,}000 \times 2 \times 312.6}{7.5} = 1{,}333{,}300 \text{ in.-lbs.}$$

which is safe for both working loads and breaking loads. The girders are safe for shear.

Note.—The batter of the posts should have been made greater so that the head sheave girders would have been shorter. The sheave girders can best be designed after the details have been partially completed.

EXAMPLES OF HEAD FRAMES.—Data for several steel head frames are given in Table XIX. Detail plans for several steel head frames are given in the latter part of this chapter.

A Timber Head Frame.*—The timber head frame shown in Fig. 83 was designed by Mr. H. L. Botsford in 1909 for the Hiawatha shaft of the Munro Iron Mining Company, Iron River, Michigan. The head frame is of the 4-post type and was designed to withstand the breaking strength of a 1¼ in. crucible steel hoisting rope with a factor of safety of three. The back brace was placed as nearly as possible in the plane of the resultant of the stresses in the hoisting rope. The main members of the back brace and the tower bents are 12 in. × 12 in. timbers. The details of the frame are shown in Fig. 83.

Steel Head Frame for the Speculator Mine.—The steel head frame for the Speculator mine, Butte, Montana, erected by the Gillette-Herzog Mfg. Co., is shown in detail in Fig. 84. This head frame is of the A-type and is 50 ft. from the collar of the shaft to the center of the sheaves, which are 7 ft. in diameter. This head frame is very rigidly braced and has given very satisfactory results. The total weight of the structural steel work exclusive of the sheaves was 42,200 lbs. At 40 cents per hour the shop labor cost 2.44 cts. per lb., an extremely high figure. The cost of erection, everything being riveted, was $12.00 per tor

Steel Head Frame for the Basin and Bay State Mining Company.—The steel head frame built for the Basin and Bay State Mining Company, Basin, Mont., by the Gillette-Herzog Mfg. Co., is shown in Fig. 85. The head frame was built on the side hill, the distance from the base of the main column to the center of the sheaves being 70 ft., while the back brace extends 10 ft. lower. The sheaves are 10 ft. in

* Engineering and Mining Journal, June 0, 1911.

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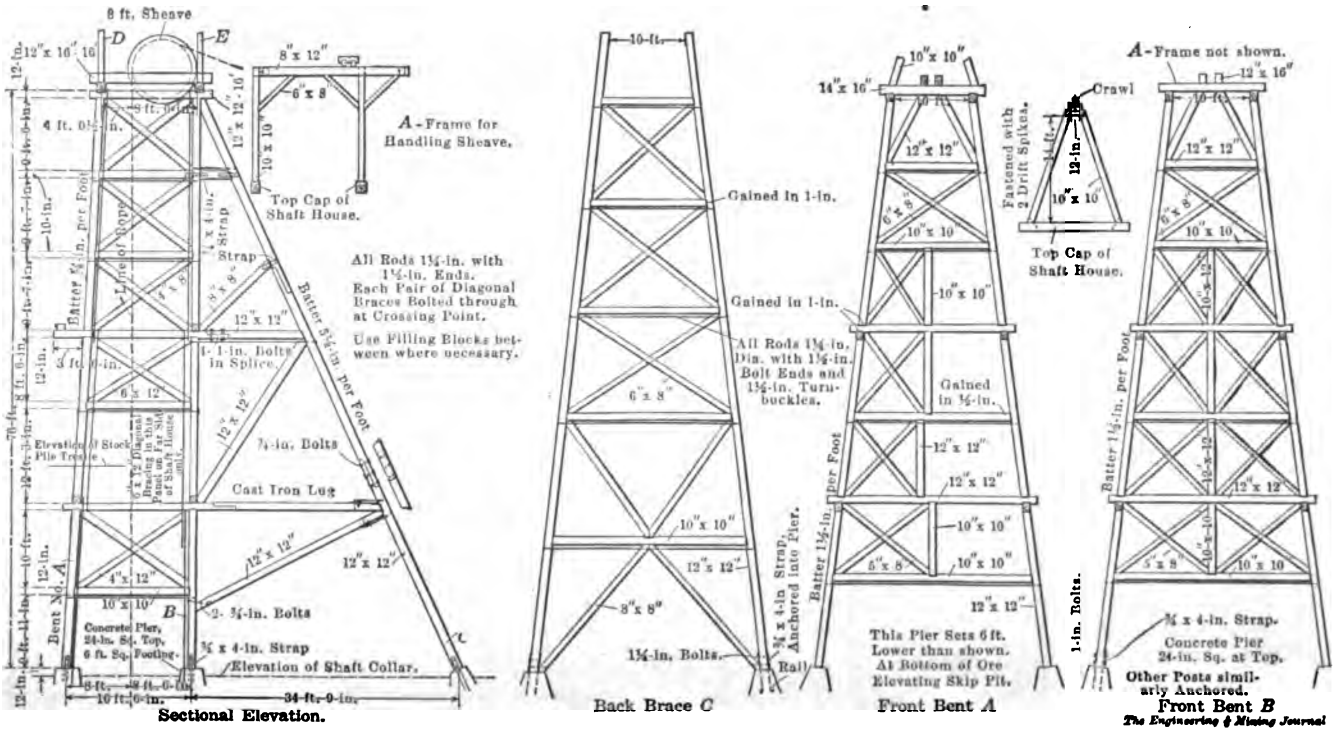

FIG. 83. TIMBER HEAD FRAME, MUNRO IRON MINING COMPANY.

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diameter and carry a steel hoisting rope 1¼ in. in diameter. The total weight of the structural steel work in this head frame exclusive of the sheaves was 79,000 lbs. At 40 cents per hour the shop cost of this head frame was 1.64 cts. per pound. The actual cost of erection, everything being riveted, was $11.20 per ton.

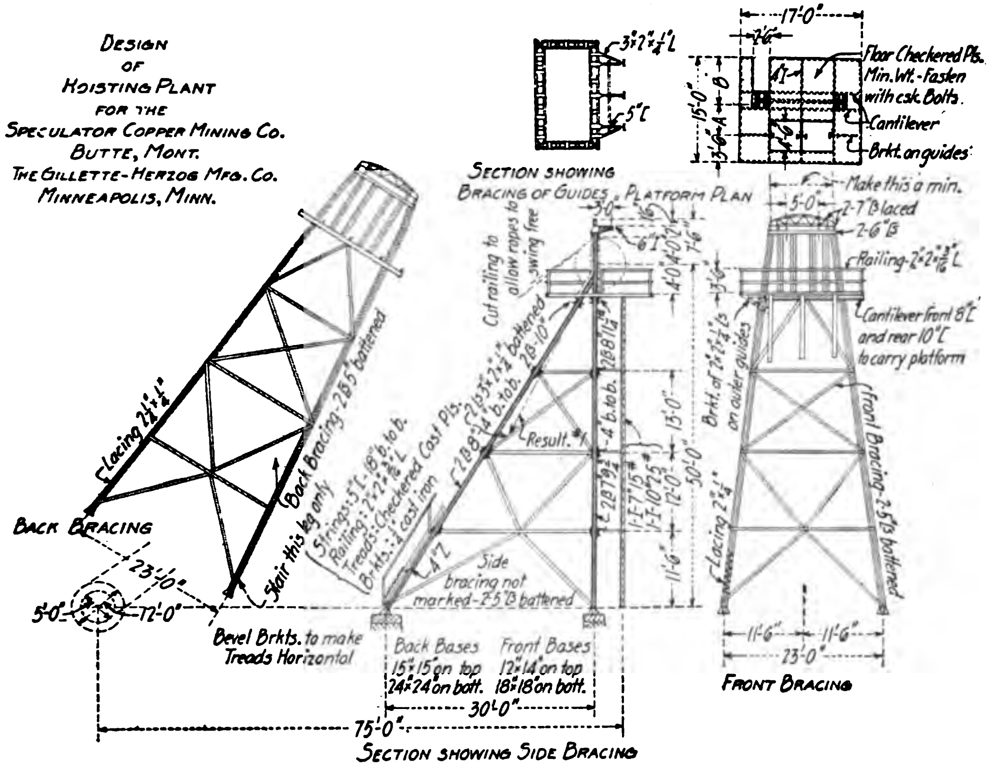

FIG. 84.

Steel Head Frame for Steward Mine.—The steel head frame built by the Gillette-Herzog Mfg. Co. for the Steward mine, Butte, Montana, is shown in detail in Fig. 86. The head frame is 55 ft. from the base to the center of the sheaves. The shaft is inclined at an angle of 71°. The ore is hoisted in steel skips carrying 5 tons. The sheaves are 7 ft. in diameter and carry a 7 in. × ½ in. flat rope. The weight of the structural steel work in this head frame is 45,000 lbs. At 40 cents per hour the shop cost was 2.08 cts. per lb. The high shop cost was probably

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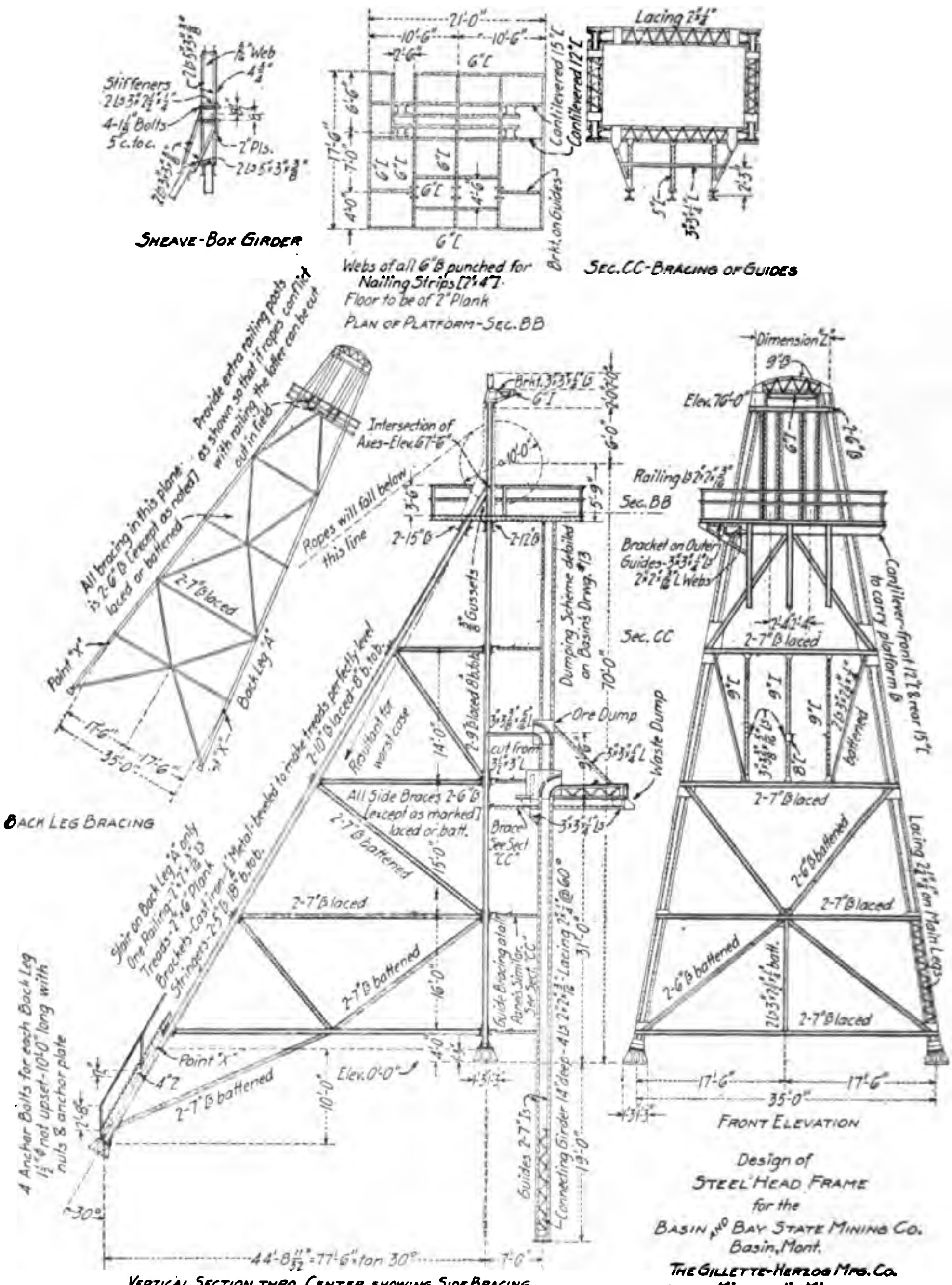
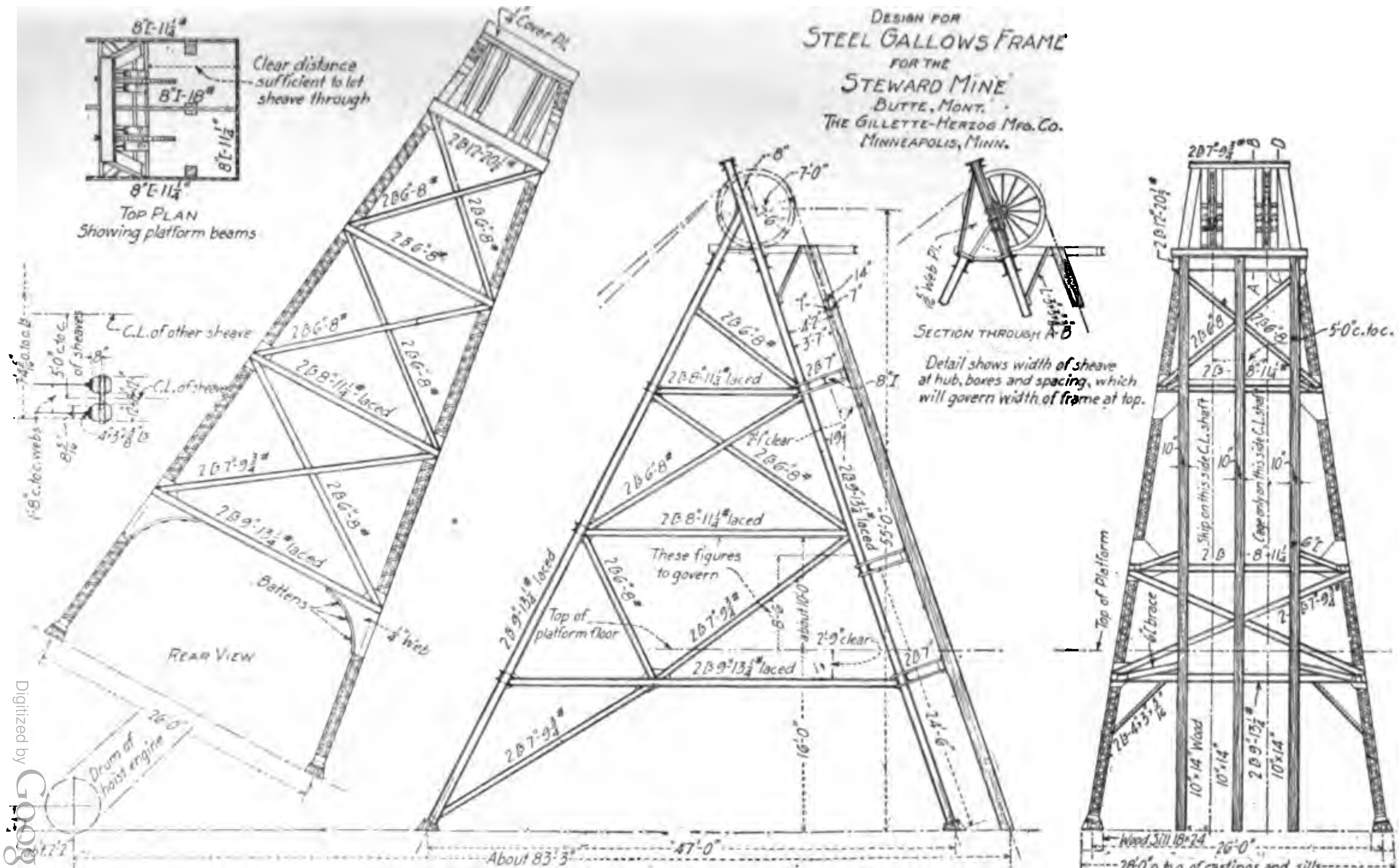

Sheave-Box Girder
Stiffeners
4-1½" Bolts 5" c. to c.
2" Pls.
Webs of all 6" B punched for Nailing Strips [2"×4"].
Floor to be of 2" Plank
Plan of Platform-Sec. BB
Lacing 2"×¼"
Cantilevered 15" C
Cantilevered 12" C
Brkt. on Guides
Sec. CC-Bracing of Guides
Provide extra railing posts as shown so that if ropes conflict with railing, the latter can be cut out in field.
All bracing in this plane is 2-6" B [except as noted] laced or battened
2-7" B laced
Point "X"
Back Leg "A"
Back Leg Bracing
Intersection of Axes-Elev. 67'-6"
Ropes will fall below this line
Sec. BB
Sec. CC
Dumping Scheme detailed on Basin's Drwg. #13
Ore Dump
Waste Dump
Resultant for worst case
All Side Braces 2-6" B [except as marked] laced or batt.
2-7" B battened
2-7" B laced
Stair on Back Leg "A" only
4 Anchor Bolts for each Back Leg 1½" Ø not upset-10'-0" long with nuts & anchor plate
Elev. 0'-0"
Guides 2-7" B
Connecting Girder 14" deep
Guide Bracing at all Panels Similar. See Sect. "CC"
44'-8 11/32" = 77'-6" × tan 30°
Vertical Section thro. Center showing Side Bracing
Dimension "Z"
Elev. 76'-0"
Railing
Bracket on Outer Guides
Cantilever-front 12" Ǐ & rear 15" C to carry platform B
Lacing 2¼"×⅝" on Main Legs
2-6" B battened
Front Elevation
Design of Steel Head Frame for the Basin and Bay State Mining Co. Basin, Mont.
The Gillette-Herzog Mfg. Co. Minneapolis, Minn.

FIG. 85.



VERTICAL SECTION C-D.

FRONT ELEVATION-SOUTH

FIG. 86.

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due to the large percentage of details. The actual cost of erection, everything being riveted, was $15.20 per ton.

Steel Head Frames for the High Ore and Diamond Mines.—The steel head frames for the High Ore and Diamond mines of the Anaconda Copper Mining Co., Butte, Montana, are essentially duplicates.

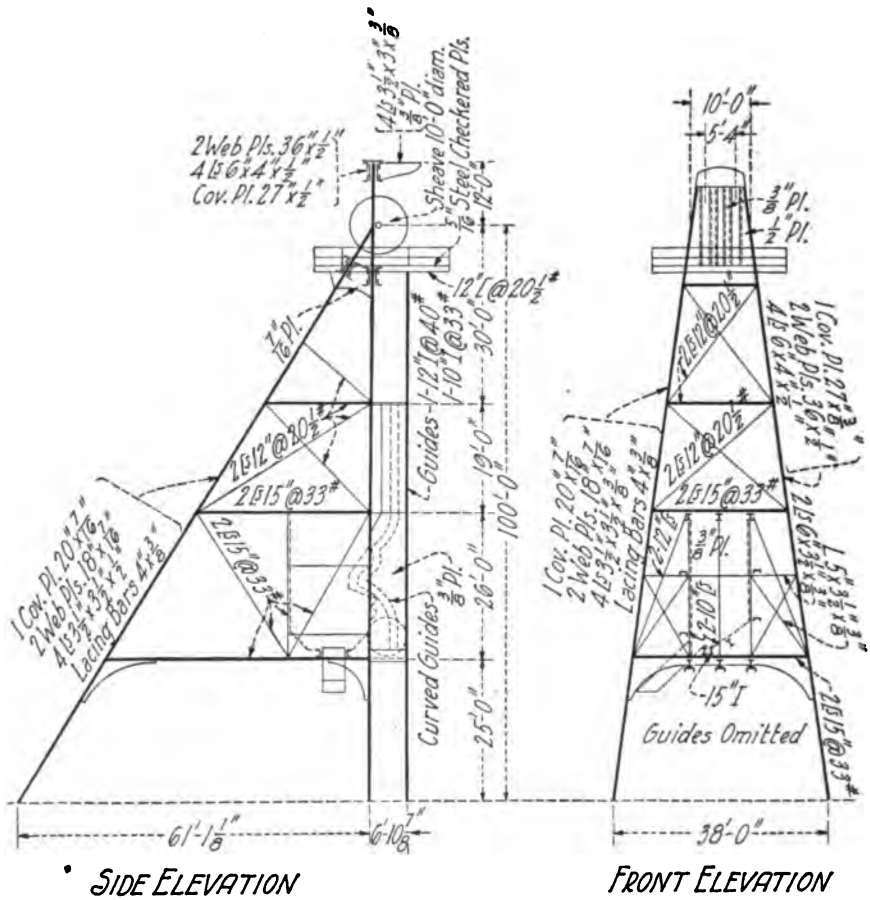

FIG. 87. STEEL HEAD FRAME FOR DIAMOND MINE, BUILT BY THE GILLETTE-HERZOG MFG. CO.

conda Copper Mining Co., Butte, Montana, are essentially duplicates. The details of the steel head frame of the Diamond mine are shown in Fig. 87. The Diamond head frame is 100 ft. high from the collar of

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the shaft to the center of the sheaves. The shaft is 2,800 ft. deep. The sheaves are 10 ft. in diameter and carry a 7 in. × ½ in. flat rope. The ore is hoisted in self-dumping skips with a capacity of 7 tons and weighing 3½ tons, and is dumped into hoppers from which it is run directly into cars which pass beneath the head frame. The main front columns and back braces are made of built-up sections consisting of one cover plate 20″ × $\frac{7}{16}$″, two plates 18″ × $\frac{7}{16}$″, 4 angles 3½″ × 3½″ × ½″, with lacing bars on the inner side 4″ × ⅜″. The main diagonal bracing is made of two channels laced. The total weight of the structural steel in the head frame proper was 292,000 lbs., while the steel work in the bins weighed 26,000 lbs. At 40 cents per hour the cost of shop labor on the structural steel was 1.09 cts. per lb. The cost of erection, everything being riveted, was $11.20 per ton.

The steel head frame at the High Ore mine is an exact duplicate of the steel head frame at the Diamond mine with the exception that the chutes and bins are omitted. A general view of the High Ore head frame is shown in Fig. 9. The High Ore shaft is 2,800 ft. deep. The total weight of the structural steel in the High Ore head frame is 292,000 lbs. Both of these head frames have proved to be very satisfactory. It would appear that equally satisfactory results could have been obtained with head frames having lighter sections—see New Leonard steel head frame, Fig. 97. The first steel head frame built in the Butte district, the Boston and Montana Copper Company's head frame at West Colusa mine, vibrated somewhat in service, and for this reason the engineers of the Anaconda Copper Mining Company decided to run no risks and specified structures made with very heavy sections.

Mr. August Christian, chief engineer, under date of October 12, 1911, wrote the author as follows: "The High Ore and Diamond head frames are very satisfactory. The mine gases are not corroding the head frames, which are painted with No. 400 graphite paint."

Steel Head Frame for the Inland Steel Company.—The steel head frame built for the Inland Steel Company at Hibbing, Minnesota, is of the A-type and has a height of 76 ft. 0 in. from the base to the center of the sheaves. The sheaves are 6 ft. in diameter and carry a crucible steel rope 1⅛ in. in diameter. The shaft has three compartments with one compartment directly behind the other two. The shaft is vertical and hoisting is done with Kimberley skips. From the center of the shaft to the center of the hoisting drum is 150 ft. The Kimberley skips weigh 3,700 lbs. and carry a load of 6,700 lbs., making a gross load of

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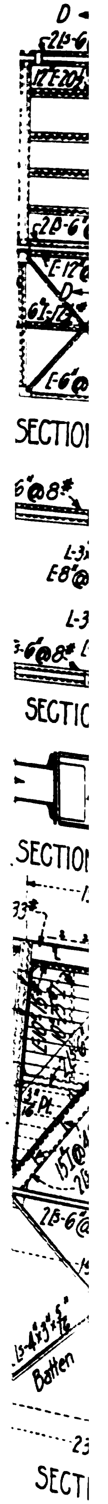
FIG. 88. STEEL HEAD FRAME FOR INLAND STEEL COMPANY, HIBBING, MINN.

10,400 lbs. The present depth of the mine is 225 ft The iron ore weighs 110 lbs. per cu. ft. A general view of the head frame is shown in Fig. 88, and detail drawings are shown in Fig. 89. This frame was



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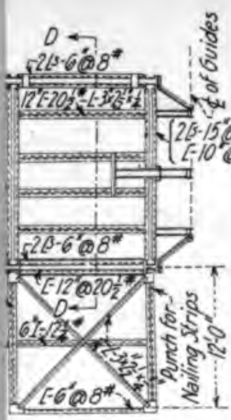
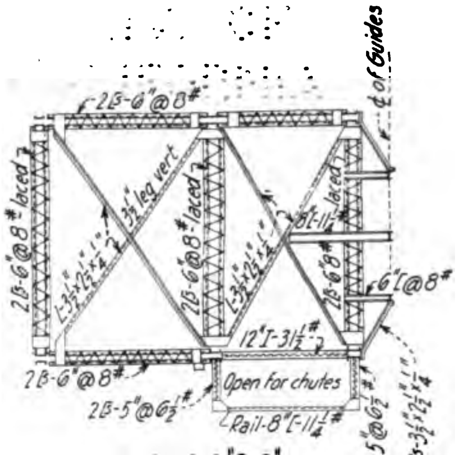
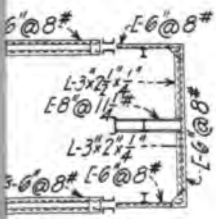


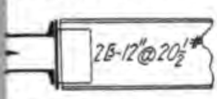
SECTION "B-B"



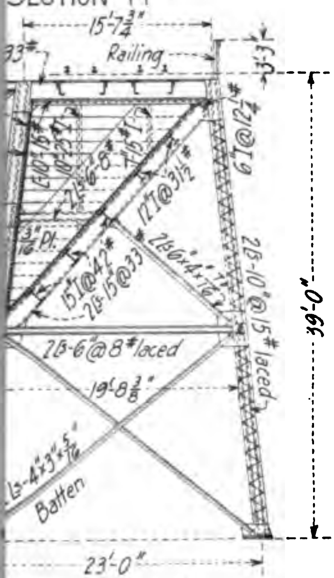
SECTION "A-A"



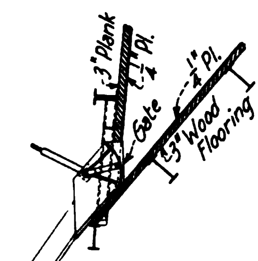
SECTION "M-M"



SECTION "Y-Y"



SECTION "GG"



SECTION THROUGH GATE OPENING

DATA:-

Rope 1⅛" Crucible Steel.
Weight of Skip 3700 lbs.
" " Load 6700 lbs.
Center of Hoist 150 ft. from ¢ Shaft.

GEN. NOTE:-

Paint one coat of mineral in shop and one coat after erection.

ERECTION PLAN

HEAD FRAME AND 100 TON ORE BIN

FOR

INLAND STEEL CO.

HIBBING,—MINN.

ORD. #22000

MINNEAPOLIS STEEL AND MACHINERY CO.

MINNEAPOLIS, MINN. - OCT. 1908.

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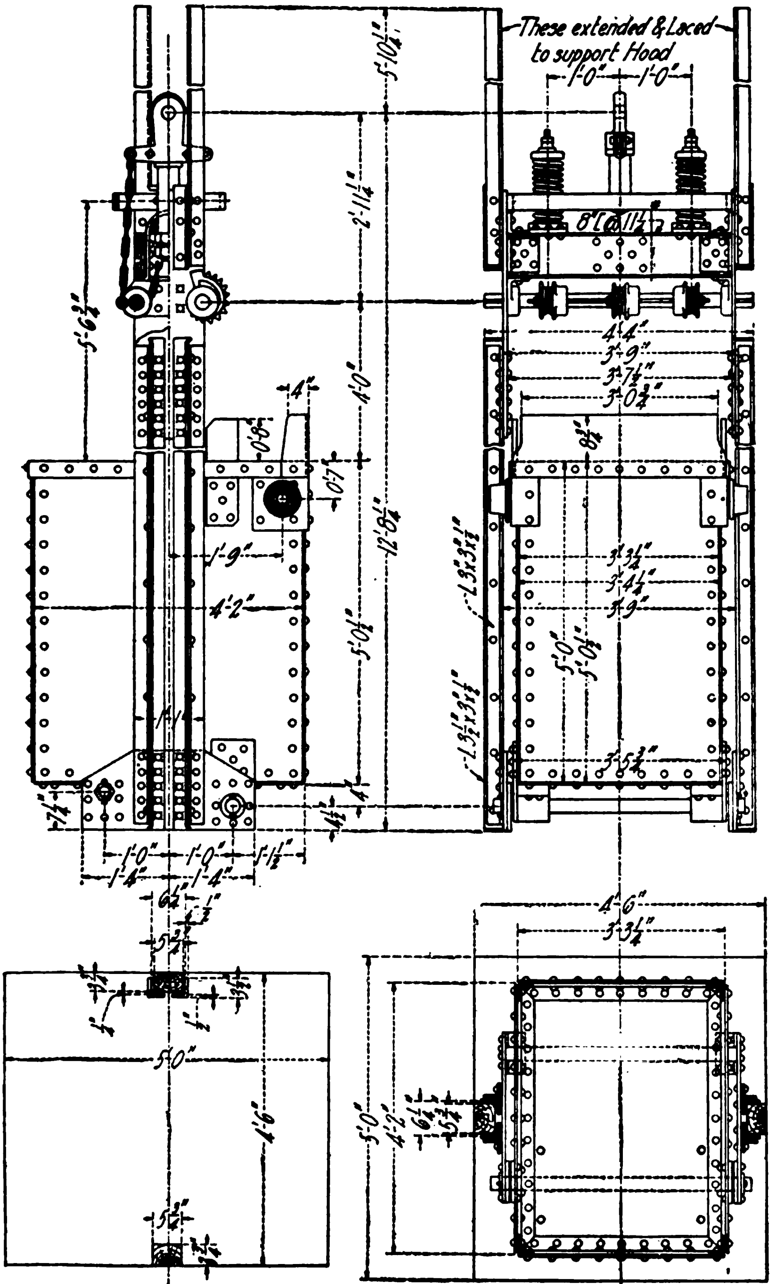

FIG. 91. SKIP FOR COPPER QUEEN CONSOLIDATED MINING CO. (123)

designed by the Minneapolis Steel & Machinery Company, Minneapolis, Minn., and was built under Standard Specifications for Materials and Workmanship. The shipping weight of the structural steel for the Inland head frame was 79,000 lbs. The steel work was painted with one coat of paint in the shop and one coat after erection. The author

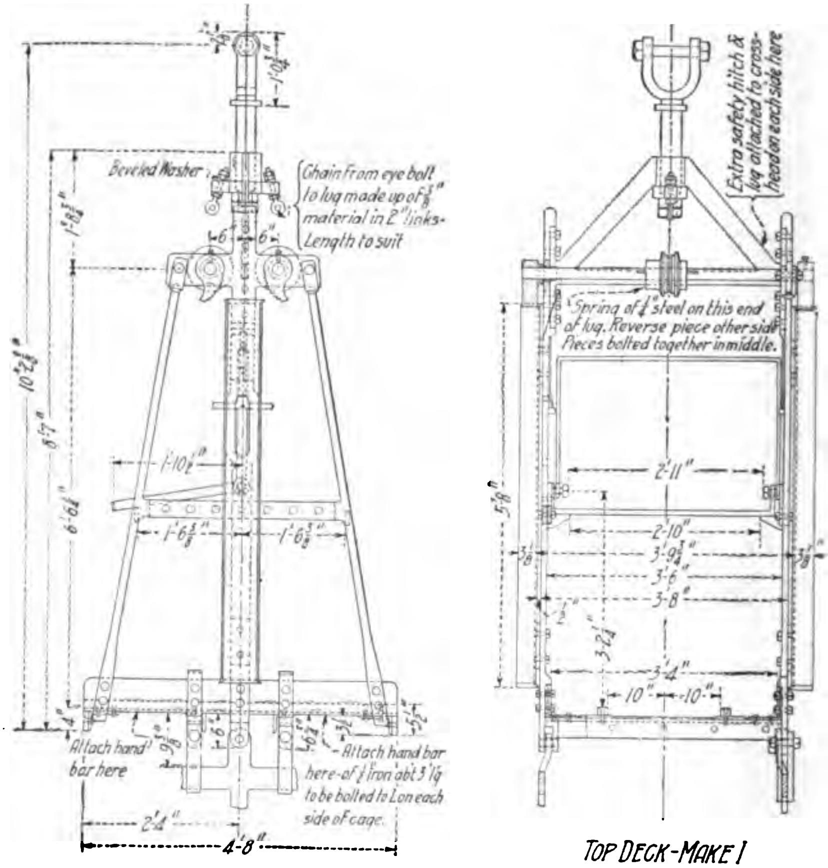

FIG. 92. TOP DECK, THREE DECK CAGE FOR COPPER QUEEN CONSOLIDATED MINING CO.

is under obligations to Mr. Wm. Werne, superintendent, for data and information.

Steel Head Frame for the Copper Queen Consolidated Mining Company.—The steel head frame erected by the Minneapolis Steel &

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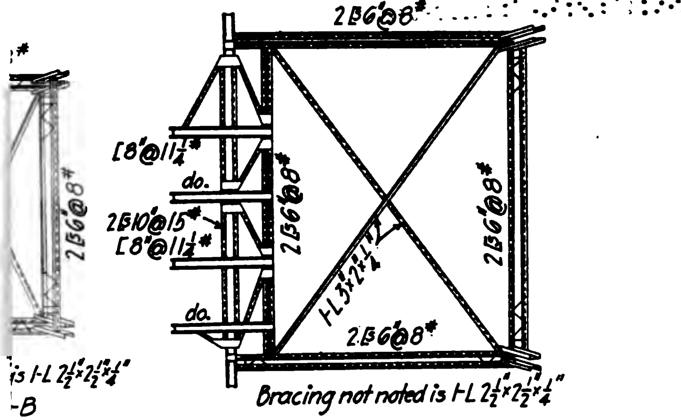
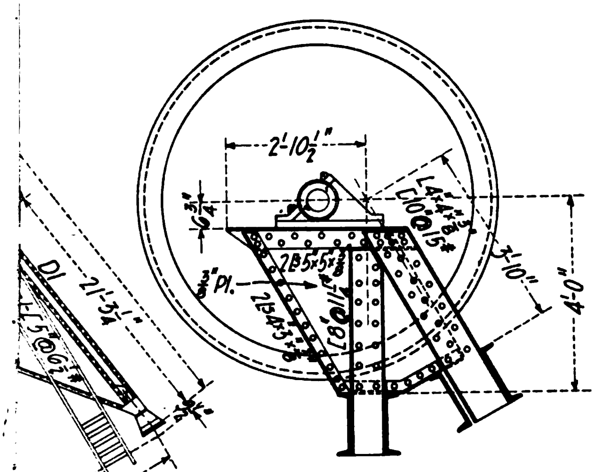


SECTION GC



ENLARGED SECTION SHOWING SHEAVE SUPPORTS

ERECTION PLAN
FOR
GALLOWS FRAME-SACRAMENTO SHAFT
COPPER QUEEN CONSOLIDATED MINING CO.
BISBEE, ARIZONA
MINNEAPOLIS STEEL & MACHINERY CO.,
MINNEAPOLIS, MINN.

...s riveted except as noted
... Coat Goheen's Carbonizing
... after erection
...ccessible after erection to
...cond coat before erection

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Machinery Company at the Sacramento shaft of the Copper Queen Consolidated Mining Company, Bisbee, Arizona, is shown in Fig. 90. This head frame is of the A-type and is 60 ft. 0 in. from the collar of the shaft to the center of the sheaves. The sheaves are 7 ft. in diameter and carry a steel hoisting rope $1\frac{1}{4}$ in. in diameter. The shaft is 1,700 ft. deep, the hoisting at present being from a depth of 400 to

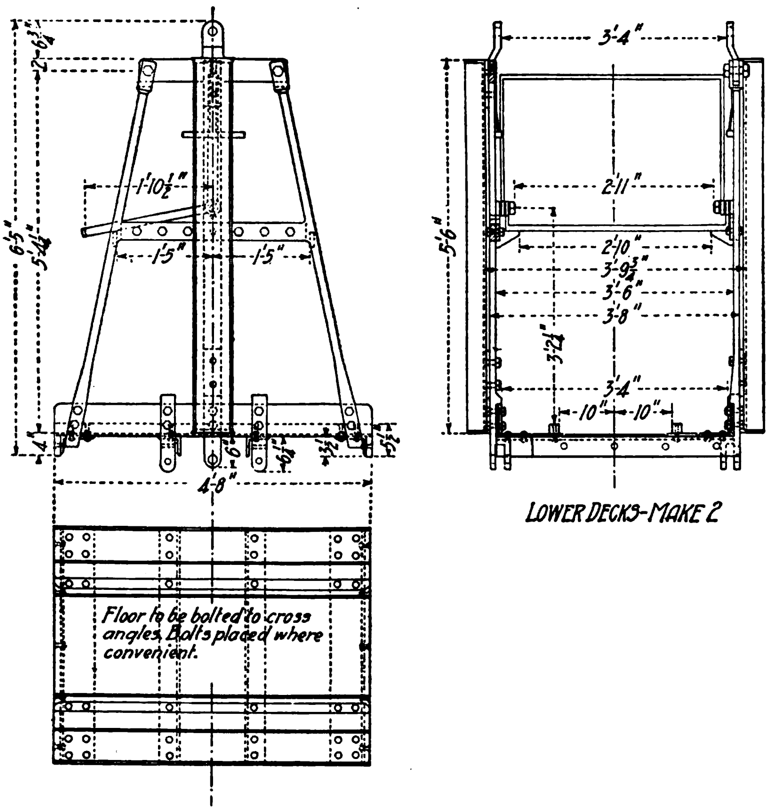

FIG. 93. LOWER DECK, THREE DECK CAGE FOR COPPER QUEEN CONSOLIDATED MINING CO.

1,600 ft. The hoisting is done in balance, the tonnage varying from 1,800 to 2,000 tons dry weight per day of 16 hours. The weight of the skip with hood is 5,990 lbs., while the weight of a load of wet ore is 3,700 lbs. The hoisting drums are 7 ft. in diameter, and are placed so that the distance from the center of the drum to the center of the

sheave is 120 ft. The hoisting rope has two laps on the drum. The weight of the structural steel in the head frame, exclusive of the sheaves, is 35,250 lbs.

Skip.—The skip used by the Copper Queen Consolidated Mining Company is shown in Fig. 91. The skip is of the Kimberley type. The details of the skip are shown in the cut.

Cages.—The standard three deck cage is shown in Fig. 92 and Fig. 93. The details are shown in the cuts.

After this head frame was completed the hoisting drum was moved farther away from the shaft, and an extra back brace, similar to that shown in Fig. 90, was built to give greater rigidity.

Tonopah-Belmont Steel Head Frame.—The Belmont shaft of the Tonopah-Belmont Mining Co., Tonopah, Nevada, is at present 1,420 ft. deep. It has three compartments, one for the ladder-way and pipes and two for hoisting. Double-deck cages of the Leadville type are used for hoisting, but the use of skips is contemplated later. The head frame, Fig. 94, is of the A-type, and the height is 75 ft. from the base to the center of the sheaves. The hoisting drum is placed 100 ft. from the center of the shaft.

The sheave wheels are of the bicycle pattern with a diameter of 84 inches at the center of the rope groove, and an over all diameter of 91 in. Each wheel has 16 spokes of 1⅛ in. rolled iron rods. The spokes are cast at their inner ends into two rings 16 in. in diameter and 3 in. wide, so that they form integral parts of the hub, which is 12 in. in diameter and 16 in. long, while the outer ends are cast into bosses on the inside of the ring. The rolled steel shafts are 16 in. in diameter at the central portion with bearings 5 in. in diameter, and are 12 in. long. The rope grooves are turned in hard maple blocks fastened in a recess in the rim. The total weight of the sheaves is 2,950 lbs. each.

The head frame is designed so as to give a factor of safety of 8 when there is on each sheave a load of 100,000 lbs. The head frame is sufficiently strong and rigid to permit of hoisting loads of 7 tons from a depth of 2,000 ft. at a speed of 1,000 ft. per minute without appreciable vibration during the most severe period of starting and acceleration.

The head frame was built by the Koken Iron Works, St. Louis, Mo., was made of structural steel furnished under standard specifications, and was fully riveted up in place with pneumatic hammers. The shipping weight of the structural steel was 63,000 lbs.

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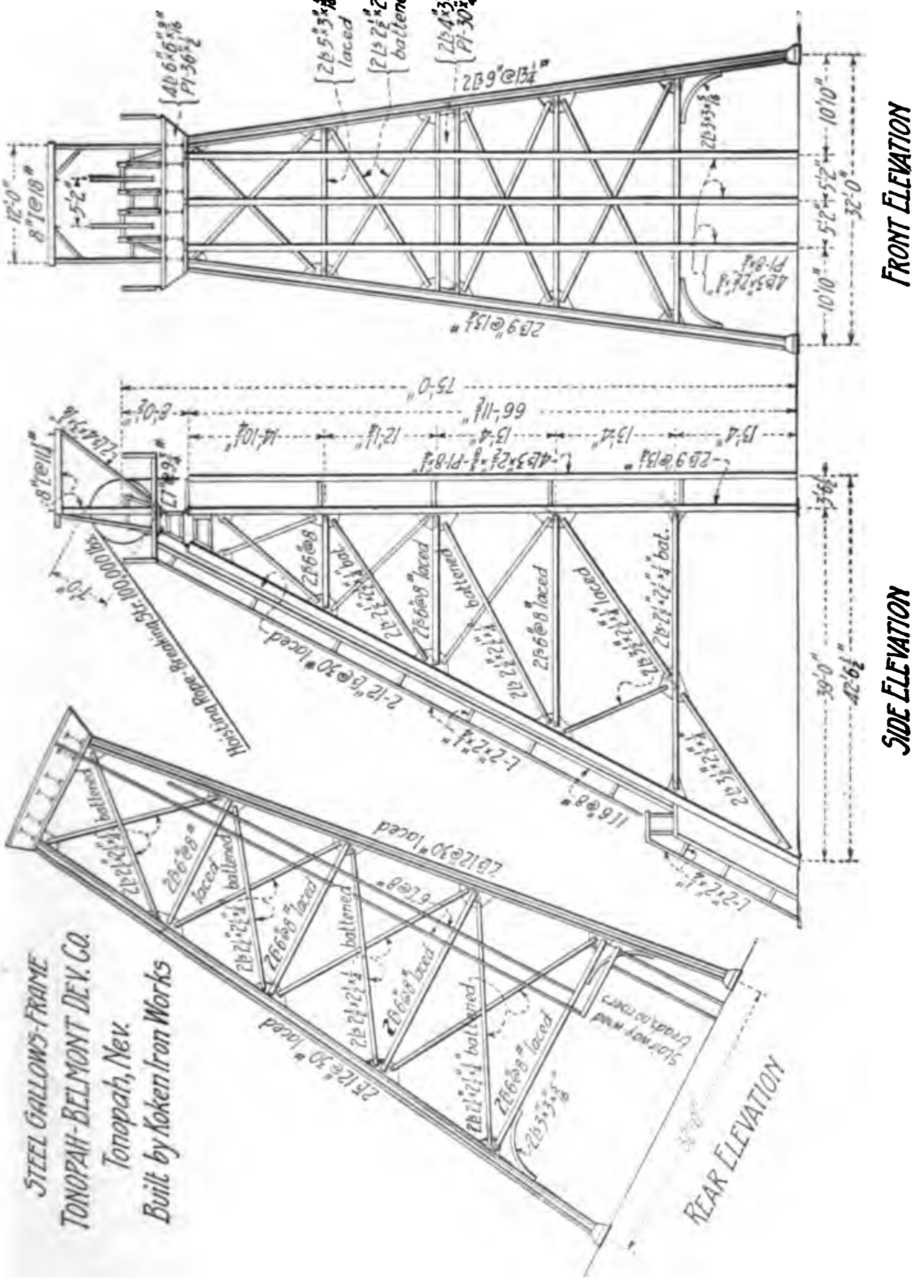
STEEL GALLOWS-FRAME
TONOPAH-BELMONT DEV. CO.
Tonopah, Nev.
Built by Koken Iron Works
FRONT ELEVATION
SIDE ELEVATION
REAR ELEVATION
75'-0"
66'-11"
32'-0"
Hoisting Rope-Breaking Str. 100,000 lbs.

FIG. 94.

The hoist is placed 100 ft. from the shaft, and is a Wellman-Seaver-Morgan double drum electric hoist with drums having 64 in. diameter and a face 36 in. wide between flanges. The hoist is designed to operate in or out of balance and is capable of handling a load of 12,000 lbs. at a speed of 1,000 ft. per minute. The hoisting rope is a six strand, nineteen wire, plow-steel rope, 1 in. in diameter that weighs 1.58 lbs. per ft., and each rope is 1,700 ft. long. The diameter of the drum at the hoist is 64 in., but the rope winds twice around the drum, so that the diameter is 66 in. near the end of the lift. With proper allowance for bending stresses the working stresses under the most severe conditions do not exceed the working load of 7.6 tons as given by the manufacturers of the wire rope.

Mine Structures.—The hoisting house is 36 ft. long, 24 ft. wide, with an ell at one corner 16 ft. by 9 ft. The building is without windows in the walls save for the one in the west wall of the ell. The roof has two saw teeth, the front one is 24 ft. in span and the rear one 12 ft., measured with the length of the building. The front of the rear saw tooth is taken up entirely by windows, giving a glazed area of 140 sq. ft. The front of the forward saw tooth is carried in line down to the ground level for a space 10 ft. wide in the central part of the main building, so that a space of 10 ft. wide and 16 ft. high is of glass. This gives the hoisting engineer an unobstructed view of the head frame from the collar of the shaft to the top. At night the frame is illuminated by a 3,000 c. p. regenerative flaming arc lamp fixed above the sheaves. The interior of the hoist house is kalsomined in white, and the north lighting from the saw tooth windows, with an almost entire absence of shadows, makes the best possible working conditions for the operator. The hoist house has a structural steel framework covered with cement plaster walls. The reinforcing of the cement walls is Hyrib, an expanded metal with stiffened rigid ribs. The expanded metal is No. 26 gage on the roof and No. 28 gage for the side walls. The steel lathing is carried on studding with 5 ft. centers. The roof of the hoist house consists of 1:2:4 concrete 3 in. thick, reinforced with Hyrib. After this layer had dried the upper and lower surfaces were plastered with a cement mortar composed of 5 parts cement, 12 parts sand and 1 part hydrated lime, mixed with chopped fiber and was troweled on in a layer ¾ in. thick. The finished roof is 4½ in. thick and has shown no cracks under severe conditions of temperature. The side walls are 2½ and 2¾ in. thick and were made by plastering both

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sides of the Hyrib with a mortar of 5 parts cement, 12 parts sand and 1 part hydrated lime, to which some cut fiber was added as before.

Steel Head Frame, Cia. Minera de Penoles.—The steel head frame built by the Wellman-Seaver-Morgan Company for the Cia. Minera de Penoles at Bermejillo, Mexico, is shown in Fig. 95 The steel head frame is of the A-type, and is 90 ft. from the collar of the shaft to the center of the sheaves. The sheaves are 7 ft. in diameter and carry a 1⅛ in. hoisting rope. The head frame is provided with two sheaves designed to hoist in a two compartment shaft, and arranged to operate self-dumping skips and cages. The vertical steel guides support wood guides on which the cages and skips operate At the top of the head frame and surrounding the sheaves is a platform covered with steel plate and provided with angle iron hand-railing. A ladder is attached to one of the front posts of the head frame, and extends from the ground to the sheave platform. The shipping weight of the structural steel in the head frame was 80,000 lbs.

Design of Head Frame.—The head frame is designed for a working load of approximately 17,000 lbs., consisting of a 70 cu. ft. skip loaded with ore, a cage, and 1,000 ft. of 1⅛ in. rope. The head frame was designed to withstand the stresses coming upon the structure with the following factors of safety: (1) Breaking strength of 1⅛ in. hoisting rope with a factor of safety of two; (2) a working load of 17,000 lbs. with a factor of safety of five; (3) the dead load with a factor of safety of four; (4) the wind load with a factor of safety of three. The general design and sections of the head frame are shown in Fig. 95. The posts are anchored to the foundation by four 1⅛ in. anchor bolts provided with anchor plates at the lower ends. The ends of the posts resting on the base plates are milled. The structural steel was medium steel furnished under American Manufacturer's Standard Specifications. The steel frame work was painted with one coat of graphite paint before shipment, and one coat after erection.

Cages.—Each cage was made of structural steel, the sides being made of steel plates stiffened at the edges by means of angles, and at the center by means of a continuous bar extending up one side of the cage over a steel casting at the top, and then down over the other side. At the upper end of the frame there is provided four cast steel safety catches to hold the cage if the rope should suddenly break. Each cage is also fitted with a spring draw-bar to protect the cage and rope against the sudden starting of the load. The steel plate hood is provided with

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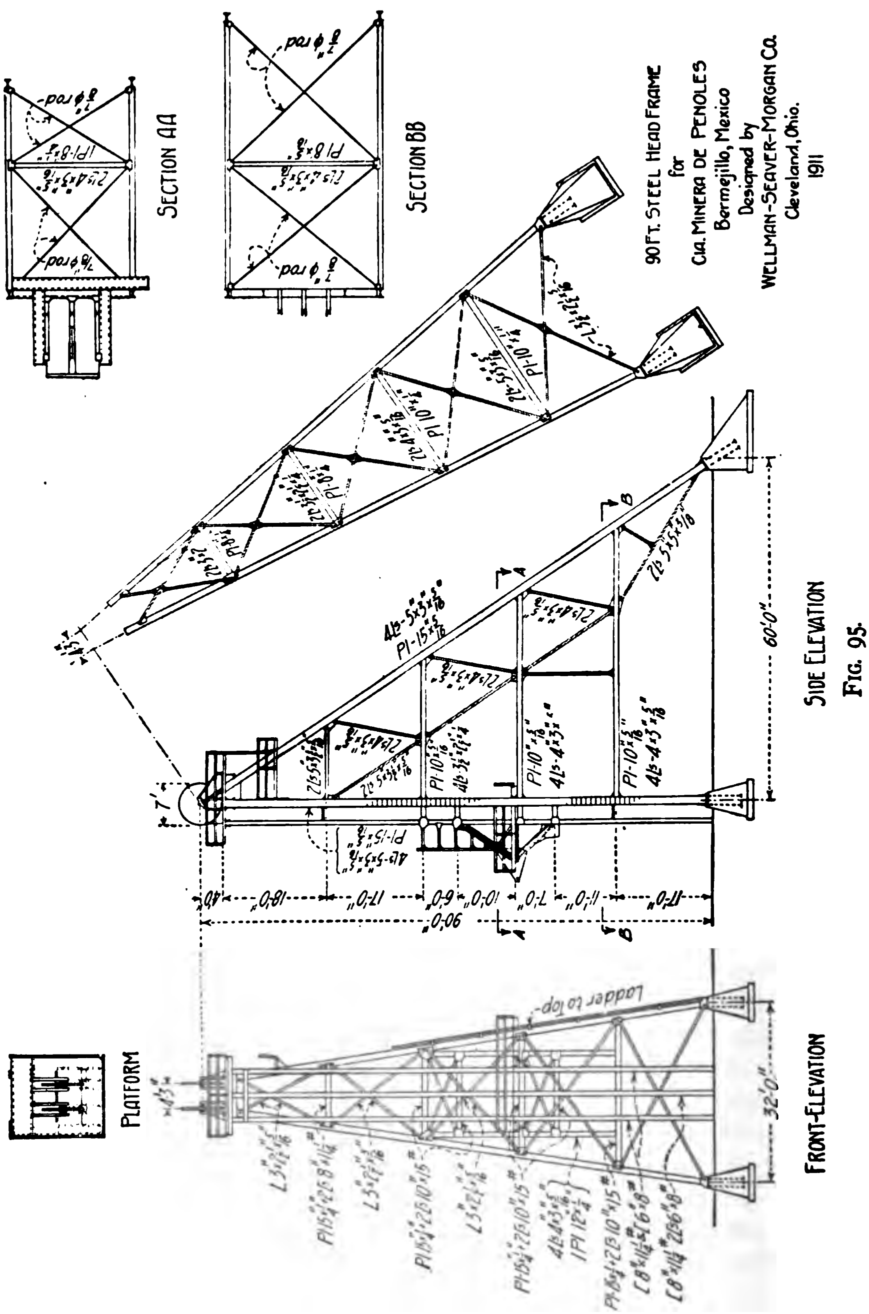
SECTION AA
SECTION BB
90 FT. STEEL HEAD FRAME
for
CIA. MINERA DE PENOLES
Bermejillo, Mexico
Designed by
WELLMAN-SEAVER-MORGAN CO.
Cleveland, Ohio.
1911
SIDE ELEVATION
60'-0"
90'-0"
PLATFORM
FRONT-ELEVATION
32'-0"
Ladder to top

FIG. 95.

hinges so that the hood may be lashed back, when lowering rails or timber. The floor of the cage is of matched oak

Skips.—The skips are made of structural steel, are 4′ 3″ × 2′ 0″ × 8′ 0″, and have a capacity of 70 cu. ft. The bottom, sides and end of the skip are fitted with a steel wearing plate bolted in place, so it can be removed when worn. The skip is provided with a lip 10 in. wide, so that the ore will dump away from the shaft as far as possible. The skip is suspended low enough under the cage so that there will be no danger of interfering with the cage in dumping. The skip is hinged at the lower left hand corner to the guide-frame, and is held in an upright position by resting on an I-beam which forms part of the guide frame. The skip is provided with two cast steel rollers at the upper left hand corner. By sliding between dumping rails these rollers guide the skip into the dumping position. At the same point there is also provided two steel plate horns, one on each side. When the skip nears the dumping position these horns slide on cast steel rollers supported on steel castings, which form part of the dumping rails. The horns also act as lifters for raising the skip onto the curved portion of the dumping rails. The sides of the guide frame are made of 3½″ × 3″ × ⅜″ angles, securely riveted to a ⅜ in. plate. These angles form guide-shoes for receiving the guide timbers in the shaft. On the inside of these angles are provided steel working plates securely bolted to the angles by counter-sunk head bolts. Two of these guide-shoes are used on the skip, and two on the guide. The frame is widened at the bottom, the left hand side receiving the hinge for the skip, and the right hand side being fitted with an I-beam for holding the skip in a vertical position.

Dumping Rails.—The dumping rails or guides consist of 3½″ × 3½″ × ⅜″ angles, held together by plates riveted to the backs of the angles. At the upper ends of the angles are provided steel castings, on which are carried cast steel rollers for receiving the horns on the skips.

Safety Detaching Hooks.—The safety detaching hooks used on this head frame are shown in Fig. 40, and are described in Chapter II.

Sheaves.—The sheaves are 7 ft. in diameter, and consist of a cast iron rim and hub, fitted with 24 wrought iron spokes 1⅛ in in diameter, cast in place. The sheave is keyed to the shaft, and has a turned groove for a 1⅛ in. rope. Each sheave has two cast iron bearings 5½ in. in diameter by 10 in. long, lined with babbitt metal and fitted with a cap, which is provided with an oil box. The upper shaft for the sheave

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is of forged steel $5\frac{1}{2}$ in. in diameter through the bearings, and 6 in. in diameter through the sheaves.

Steel Head Frame for the New Leonard Mine.—The steel head frame shown in Fig. 96 and Fig. 97 was built by the American Bridge Company for the New Leonard mine of the Boston & Montana Copper Company, Butte, Montana. The head frame is of the A-type, and is

FIG. 96. STEEL HEAD FRAME FOR LEONARD MINE.

140 ft. high from the collar of the shaft to the center of the sheaves. The mine has a four compartment shaft, two of the compartments being used for hoisting ore. The mine is now 1,697 feet deep, but the head frame was designed for an ultimate depth of 3,500 feet. The ore is hoisted in five-ton self-dumping skips with a single deck cage above the skip. The skips weigh 7,500 lbs. each. Four-deck cages are used for hoisting men. The hoisting rope is $1\frac{1}{2}$ in. in diameter, a round hoisting rope being an innovation in the Butte district. The rate of hoisting is 2,800 ft. per minute. The skip ore bins have a capacity of 150 tons. From the skip ore bins the ore runs into railroad ore bins (not shown in Fig. 97), 26 ft. 9 in. wide by 150 ft. long, with a capacity

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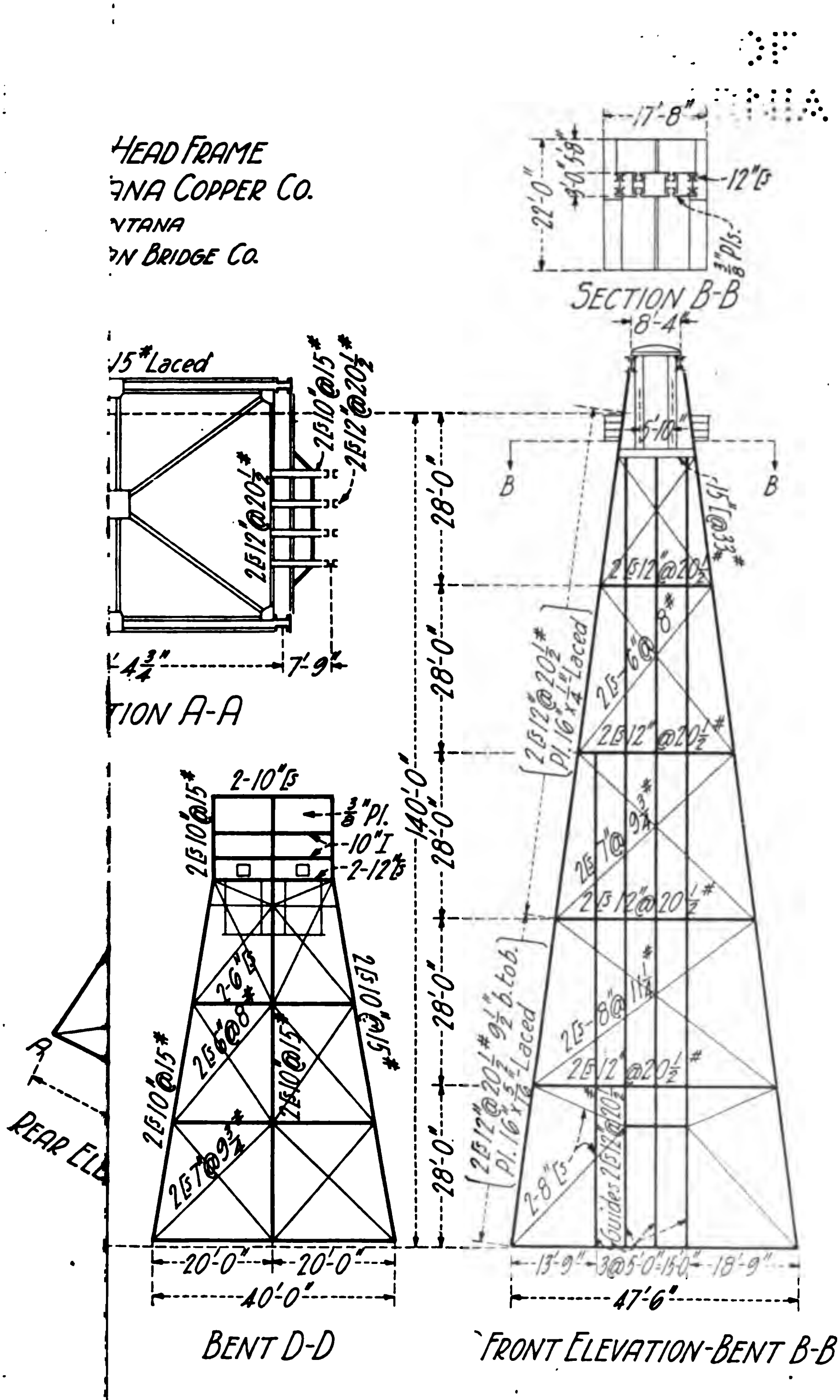

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of 1,500 tons. The sheaves are 12 ft. in diameter, and are placed 5 ft. 10 in., center to center.

The main posts are made of two channels 12 in. @ 20½ lbs., with a cover plate 16 in. wide and $\frac{5}{16}$ in. and ¼ in. thick, with lacing on the inner side. The back braces for the lower two panels are made of channels 12 in. @ 30 lbs., with a plate 16 in. × ⅜ in.; the third section is made of two channels 12 in. @ 30 lbs., with a plate 16 in. × $\frac{5}{16}$ in., while the two upper sections are made of channels 12 in. @ 20½ lbs., laced on both sides. The main struts and diagonal braces are made of two channels, with battens top and bottom. The skip guides are made of two channels 12 in @ 20½ lbs The main girder at the top of the back brace consists of one plate 36 in. × ⅜ in., and four angles 4″ × 4″ × ⅜″. The skip bins are supported on columns made of two channels 10 in. @ 15 lbs., laced on both sides. Where two channels are used for a section, the flanges are turned out. The New Leonard head frame is one of the highest in the country, and is one of the best designed frames that has been constructed. The shipping weight of the structural steel in this head frame was 346,425 lbs.

Steel Head Frame for Union Shaft.—The steel head frame shown in Fig. 98 and Fig. 98 a, was designed and erected by the Wellman Seaver-Morgan Company for the Union shaft, Virginia City, Nevada. It was impossible to obtain satisfactory foundations near the shaft opening, and it was necessary to place the front vertical legs of the head frame on a heavy box plate girder. The back braces are made of two plate girders, consisting of one plate 18″ × $\frac{5}{16}$″ and four angles 4″ × 3″ × $\frac{5}{16}$″, fully braced in the plane of the girders. The front vertical posts are built H-columns, consisting of one plate 12″ × $\frac{5}{16}$″ and four angles 3½″ × 3″ × $\frac{5}{16}$″. The posts are rigidly braced, and have a batter of 1½″ in 12″ from the bottom to a height of 40 feet. There is no bracing between the back braces and the front posts. The back braces are firmly anchored to the foundation by means of two anchor bolts in each back brace, 1½ in. in diameter by 7 ft. long. The head sheaves are 7 ft. in diameter, each carrying a 1 in. hoisting rope. The rate of hoisting is 1,000 ft. per minute, and the hoisting capacity is 500 tons from a depth of 2,000 ft.

The head frame was designed for a load of 16,000 lbs., made up as follows:

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6,000 lineal ft. of 1-in. rope	= 9,600 lbs.
2 cages	= 2,400 lbs.
2 cars	= 1,100 lbs.
Rock in car	= 2,400 lbs.
Tail sheave	= 500 lbs.
	16,000 lbs.

FIG. 98. STEEL HEAD FRAME FOR UNION SHAFT.

The vertical shaft guides are made of 8 inch I-beams @ 18 lbs. Over winding is prevented by the Wellman-Seaver-Morgan safety device shown in Fig. 40 and described in Chapter II. This consists of a detaching hook and a cast iron case attached to the head frame. This safety device is the same as that used in the steel head frame of the Cia. Minera de Penoles, Fig. 95. The weight of the structural steel in the head frame is 42,000 pounds. This head frame is very simple and economical. Under date of Nov. 16, 1911, Mr. Whitman Symmes,

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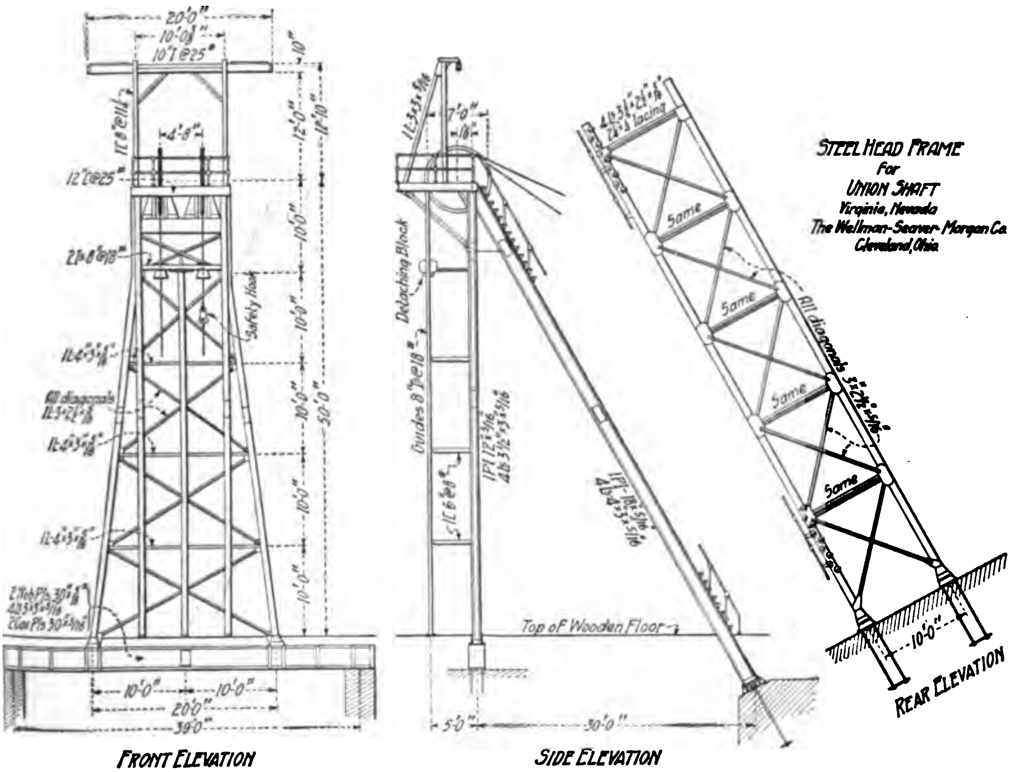
superintendent, wrote the author that the head frame was giving excellent satisfaction.

Steel Head Frame for Elkton Consolidated Milling and Mining Company.—The steel head frame shown in Fig. 99 was erected by the Wellman-Seaver-Morgan Company for the Elkton Consolidated Milling & Mining Company, in the Cripple Creek district at Elkton, Colorado. This head frame is 55 ft. from the top of the shaft to the center of the sheaves. The sheaves are 5 ft. in diameter and carry a flat



FRONT ELEVATION SIDE ELEVATION

FIG. 98 a.

hoisting rope $3\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in. The head frame is of the 4-post type; the front main vertical columns being made of one angle 6″ $\times$ 6″ $\times$ $\frac{1}{2}$″, and the middle main vertical columns being made of one angle 6″ $\times$ 6″ $\times$ $\frac{3}{4}$″. The back braces are made of two 12″ channels @ 25 lbs. The main tower is braced by side braces made of one 6″ $\times$ 6″ $\times$ $\frac{3}{8}$″ angle.

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The head frame is braced with single angles, as shown in Fig. 99. The head frame was designed for a breaking load of 76,000 lbs. in one rope, and a working load of 15,200 lbs. The hoist was designed to take an unbalanced load of 8,000 lbs., plus the weight of 2,000 ft. of $3\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in. rope This head frame, while light, is very rigid and has given excellent results. The entire head frame is enclosed in a frame building.

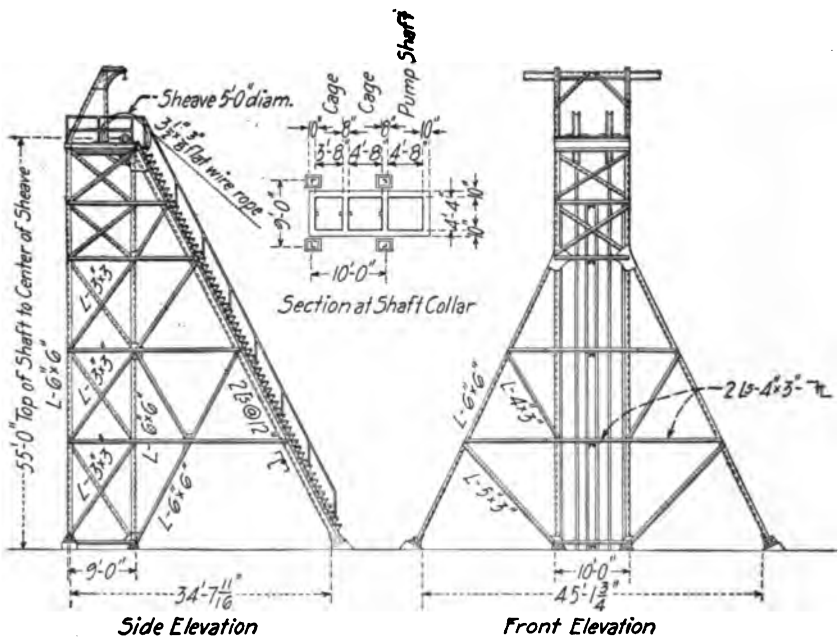

FIG. 99. STEEL HEAD FRAME FOR ELKTON MINE.

Steel Shaft House for Sibley Mine.*—The steel shaft house for the Sibley mine of the Oliver Iron Mining Company, Ely, Minnesota, shown in Figs. 100 to 103, was built by the American Bridge Company. The shaft house is 30 ft. wide by 75 ft. $4\frac{9}{16}$ in. long, and is 158 ft. from the collar of the shaft to the top of the head frame, and 140 ft. to the center of the sheaves. On account of the caving nature of the soil it was not possible to build column foundations close to the shaft, and the front columns are supported on a heavy box girder, the center line of which is 5 ft. 5 in. from the center of the shaft. The back braces are made of two channels 15 in. @ 33 lbs., laced, while

* Engineering News.

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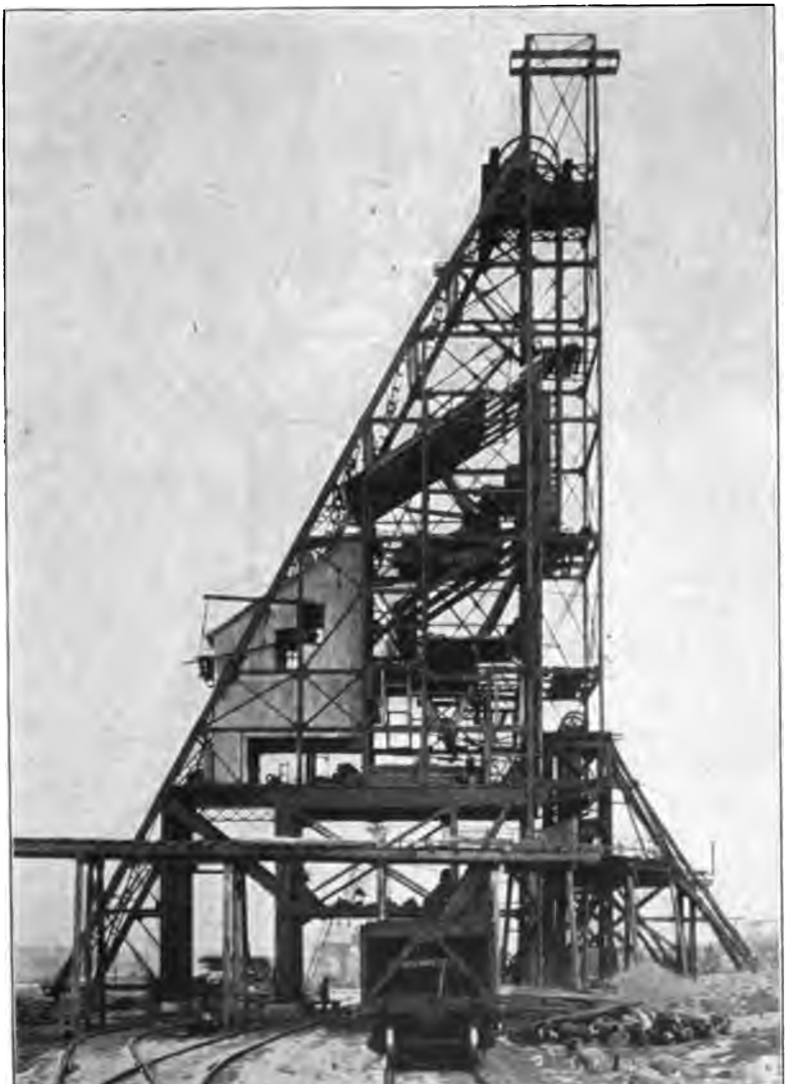

FIG. 100. STEEL SHAFT HOUSE FOR SIBLEY MINE.

the columns of the main braces are made of two channels 12 in. @ 25 lbs., laced. The head sheaves are 12 ft. in diameter and carry a hoisting cable $1\frac{3}{8}$ in in diameter. At a height of 33 ft. above the shaft there

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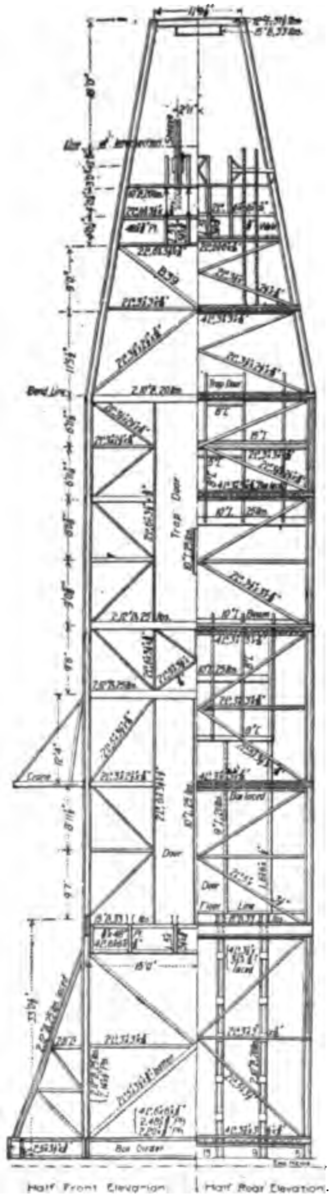

FIG. 101. STEEL SHAFT HOUSE FOR SIBLEY MINE, REAR ELEVATION.

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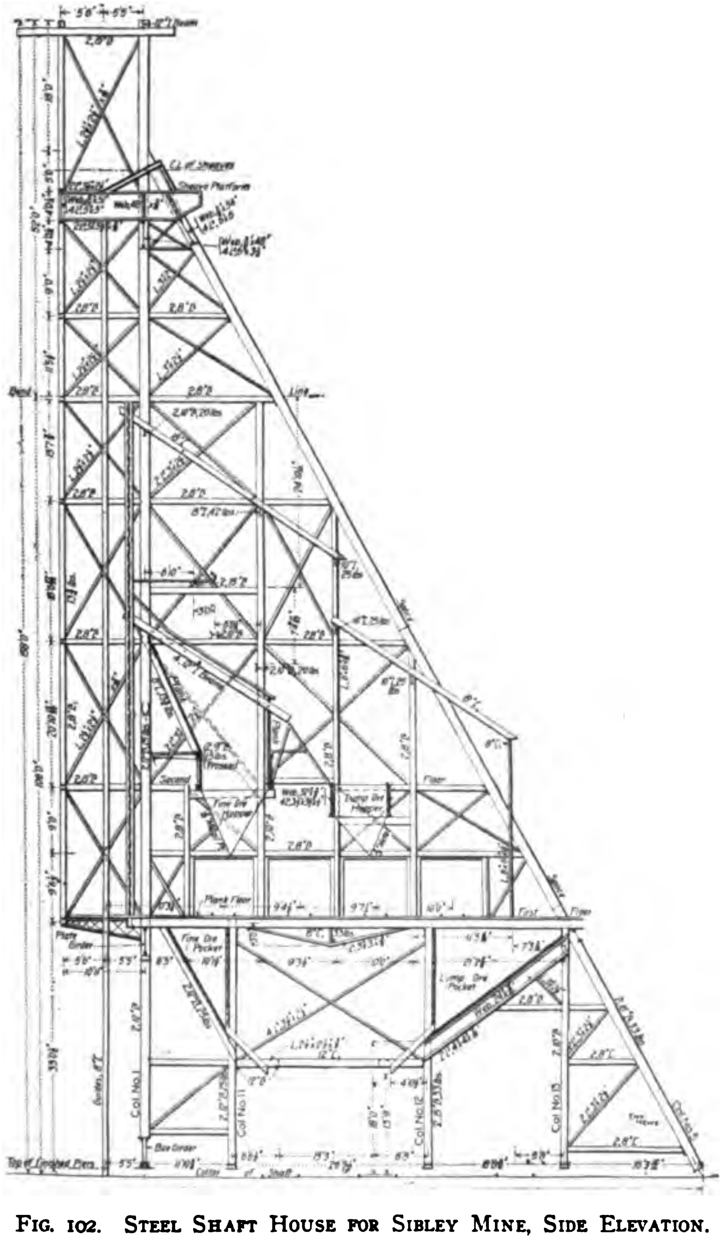

FIG. 102. STEEL SHAFT HOUSE FOR SIBLEY MINE, SIDE ELEVATION.

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is a projection on the shaft house to enclose the guides and cages. At the level of the first floor there is also a line of 36 in. plate and lattice girders between the outer columns, inside of the 15 in. channels and between the inner columns, which are shown in the sections in Figs. 101 and 102. The fine ore and the lump ore are dumped from the cages through separate hoppers into bins or pockets, from which the ore is dicharged into railway cars on the tracks which run through the shaft house. The hoppers are made of $\frac{5}{16}$ in. steel plates, while the pockets have a plank lining attached to the structural steel framework. The building is covered with corrugated steel. All field connections are riveted. All columns having $\frac{11}{16}$ in. holes at the splices were bolted together at the shops and had the holes reamed to $\frac{13}{16}$ in. diameter. The building was painted with two coats of iron ore paint after erection.

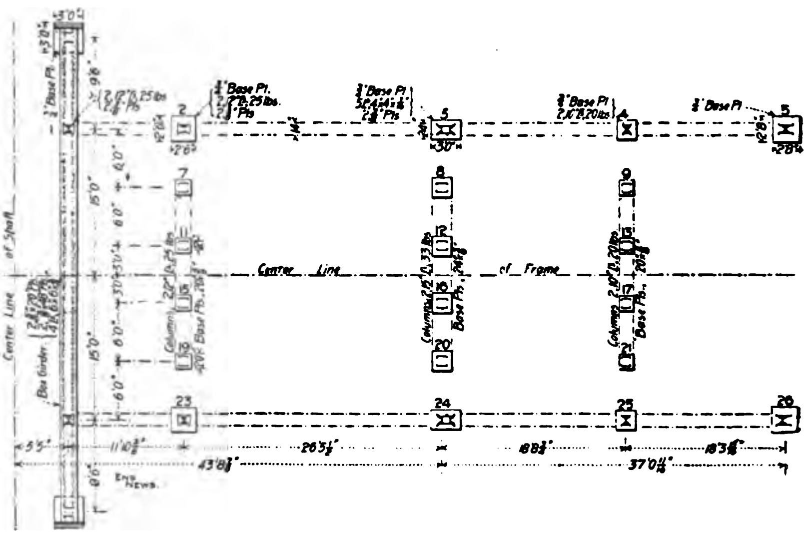

FIG. 103. FOUNDATION PLAN FOR SHAFT HOUSE, SIBLEY MINE.

When the shaft house was built, the Sibley shaft was 726 ft. deep, with hoisting compartments 5 ft. $\times$ 6 ft. The shaft house was designed for an ultimate depth of 2,000 ft The shaft is vertical. The shipping weight of the structural steel in the steel shaft house was 576,663 lbs.

Hoisting Plant.—The hoisting plant is of the type known as the twin first motion or direct acting hoist, consisting of two first motion

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hoists operated with one pair of engines. Each drum carries two ropes, two of these ropes being used in the Sibley shaft and the others

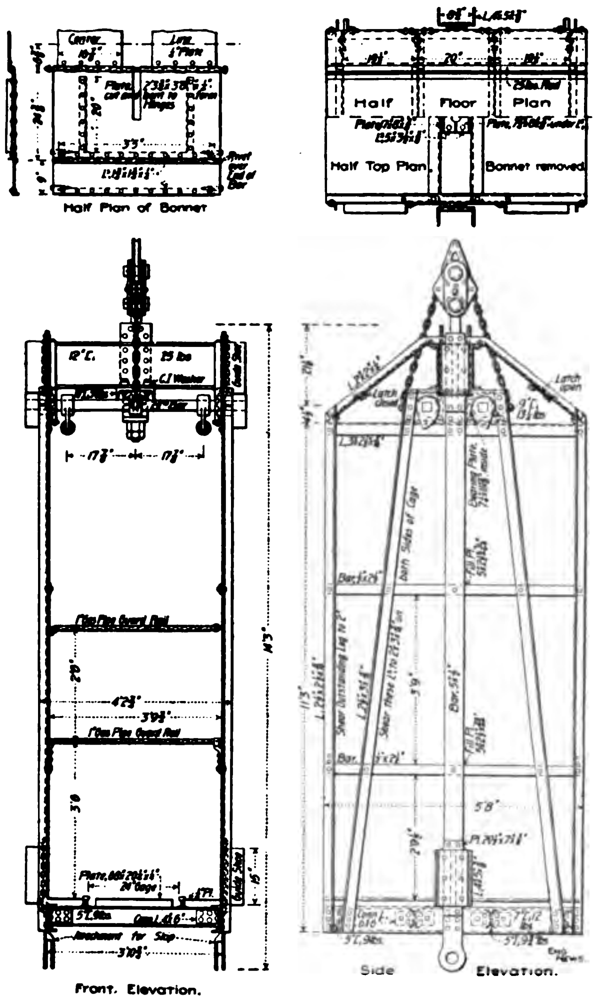

FIG. 104. STEEL CAGE FOR SIBLEY MINE.

in the Savoy shaft, which is at a distance of 1,500 ft. from the Sibley shaft. The Sibley shaft is 500 ft. from the hoisting plant, while the Savoy shaft is 2,000 ft. The ore from the tram cars is dumped under-

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ground directly into the storage pockets. When the skips are filled from the storage pockets the hoisting can be done with one hoist from

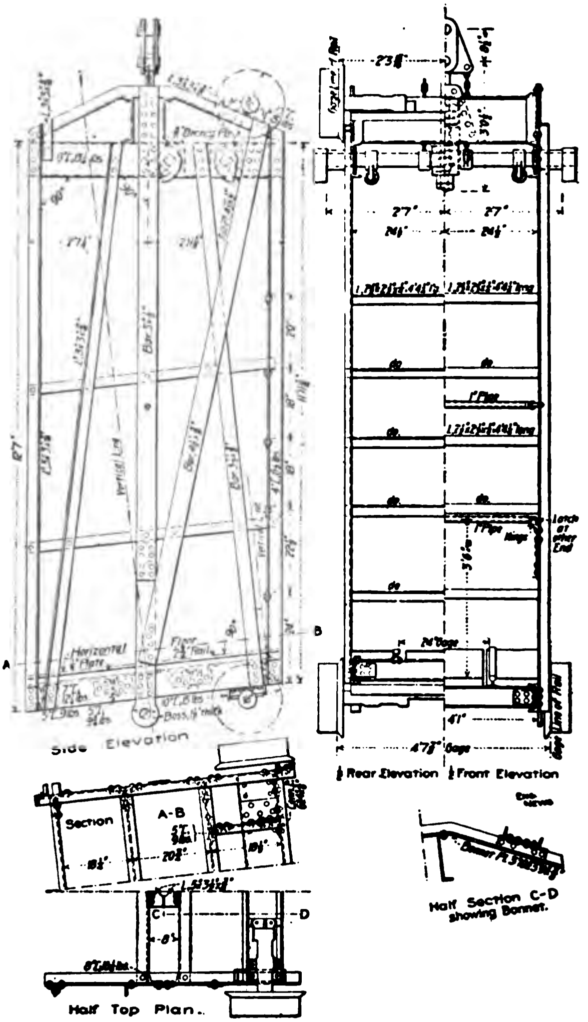

FIG. 105 STEEL CAGE FOR THE SAVOY MINE.

two different mines, without delay or waiting for the hoist in the other shaft. The hoisting capacity of each drum is 28,500 lbs., distributed as follows: ore, 14,000 lbs.; rope, 6,000 lbs.; cage, 3,500 lbs.; skip,

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5,000 lbs The unbalanced load is 21,000 lbs. The hoisting speed is 2,000 ft. per minute.

Cages.—The cages for the Sibley vertical shaft are shown in Fig. 104 and are 14 ft. 3 in. high, and 5 ft. 8 in. by 4 ft. 2½ in. in plan. The frame is made of steel angles and bars, with channels for the bottom framing. The floor is made of 2 in. planking, covered with ¼ in. steel

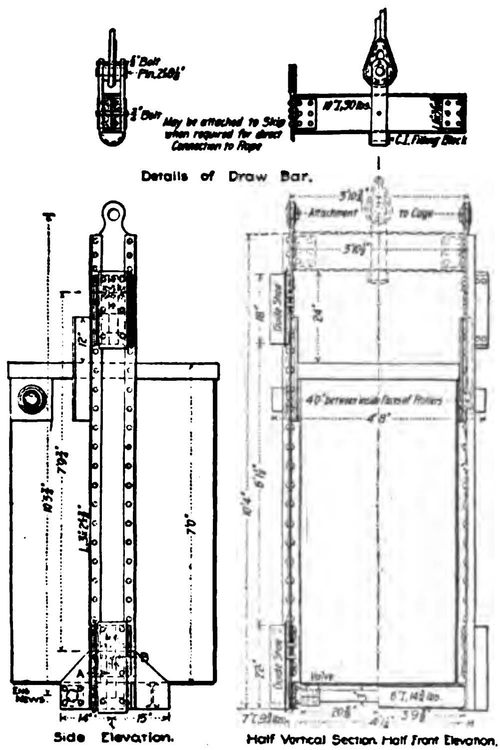

FIG. 106. STEEL SKIP FOR THE SIBLEY MINE.

plate, and between the planks are 25 lb. rails with a gage of 24 in. for the mine cars. Two heavy bars transmit the pull to the skips underneath, and at the head is a 2½ in. drawbar of soft, open hearth steel. The two drawbar springs are of square steel, and have an outside diameter of 6½ in., and a length of 7 in. when free. The compression under 10,000 lbs. load is 1½ in.

In the Savoy inclined shaft the cages run on four 18 in. flanged

wheels on a track of 4 ft. $7\frac{3}{8}$ in. gage, as shown in Fig. 105. The Savoy shaft is inclined to the vertical $1\frac{1}{8}$ in. to one foot. The framing is similar to that of the other cage but with additional diagonal members, while the lower side of the frame is made of 4 in. channels. The floor is horizontal. Attached to the safety catches are two extension springs of $\frac{7}{16}$ round steel wire.

Skips.—The skips for the Sibley shaft, Fig. 106, are 7 ft. high and are carried between two upright angle iron frames, with a total height of 10 ft. $5\frac{3}{4}$ in. The weight complete is 4,678 lbs. for a skip without lip, or 4,935 lbs. for a skip with a lip. The skips for the inclined Savoy shaft are 6 ft. 9 in. high, and run on four 18 in. wheels, like the cage. Ordinarily the skips are suspended beneath the cages, being connected to the forked end of the two heavy side bars of the cage frame by means of 3 in. pins. If the skip is at any time required to be attached directly to the rope, a cross frame with a drawbar is fitted between the side frames of the skips, as shown by the dotted lines.

Steel Rock House for the Quincy Mining Company.—The steel rock house No. 2 for the Quincy Mining Company, designed and built by the American Bridge Company, is shown in Fig. 11 and in Figs. 107 to 109.

The rock house is 170 ft. $8\frac{11}{16}$ in. by 44 ft. 6 in. in plan, with a height of 119 ft. 3 in. to the center of the head sheaves, and 146 ft. $11\frac{3}{4}$ in. to the top of the framing of the roof covering the sheaves. The shaft is a two compartment shaft, and makes an angle with the horizontal of 57 degrees. The depth of the shaft on the incline is over 6,000 ft. The hoisting capacity is 2,400 tons in 24 hours.

The shipping weight of the structural steel and corrugated steel was 839,000 lbs. The copper rock is hoisted in skips weighing 5 tons, and having a capacity of 168 cu. ft. The skips are 4 ft. by 4 ft. by 12 ft. long. After leaving the main shaft, the skips are carried on a track on the 15 lb. channels, which constitute the back brace of the head frame, and may be dumped at points 1, 2, or 3, Fig. 109. Large pieces of rock or mass copper are dumped at point 1, and are loaded directly into cars by means of a chain hoist carried on a 15 in. I-beam. The poor rock is dumped at point 2, from which it is chuted into the poor rock bin marked *C*. The poor rock from bin *C* is sorted, the rock is thrown in poor rock tube *E*, from which it is run into cars and is hauled to the waste dump. The copper rock from bin *C* is run through the 25 ton crusher, and is chuted into crusher rock bin *D*, from which

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it is run into cars. The mass rock is dumped at point 3, and runs over the grizzles, *J*, placed at an angle of 16 degrees with the horizontal. The grizzles, *J*, are made of 6 in. round bars spaced 18 in. center to center. The material passing through grizzle *J* drops into hopper bin *B*. From bin *B* the rock is thrown into crusher *M*, the poor rock being

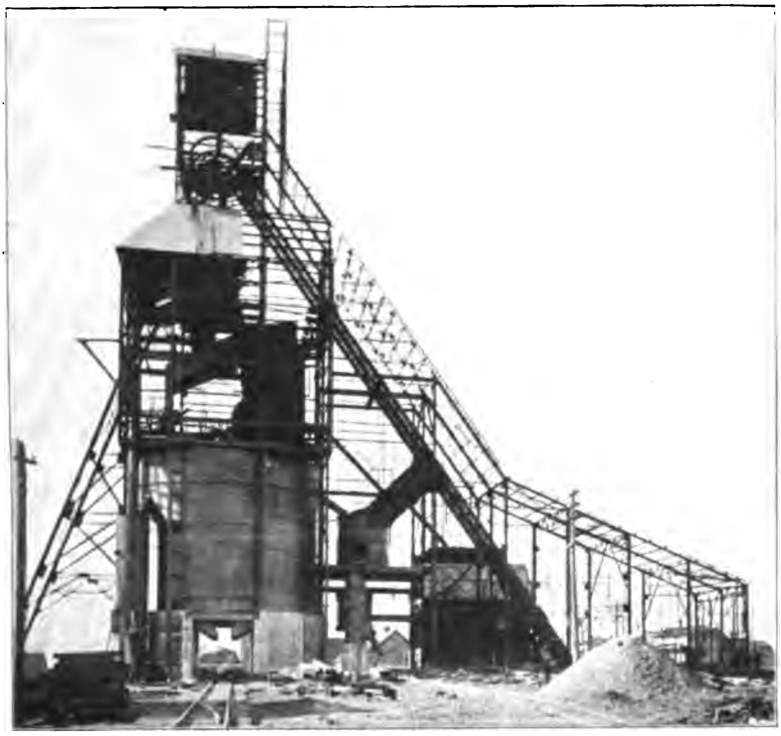

FIG. 107. STEEL ROCK HOUSE FOR QUINCY NO. 2 MINE.

sorted out and dropped through chute *K* into the poor rock bin *C*. After the rock is crushed in crusher *M*, it drops into the stamp rock bin *A*. The stamp rock is drawn from bin *A* through bottom gates into railroad cars, and is taken directly to the stamp mill.

Pieces of mass copper and large pieces of copper rock that will not pass through the grizzle *J*, pass down chute *I* into the big rock bin *H*. Masses too large to pass through mass copper chute *G* are put under the drop hammer and are reduced in size. The mass copper is dropped

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through mass copper tube *G*, and is loaded directly into cars. The copper rock is dropped into stamp rock bin *A*.

The head sheaves are 12 ft. in diameter. The hoisting cable is $1\frac{1}{2}$ in. in diameter, and makes an angle of 12° 38′ with the horizontal.

The stamp rock bin *A* is 44 ft. in diameter and 35 ft. 5 in. high. The lower 28 ft. of the bin has side plates $\frac{5}{16}$ in. thick, while the upper

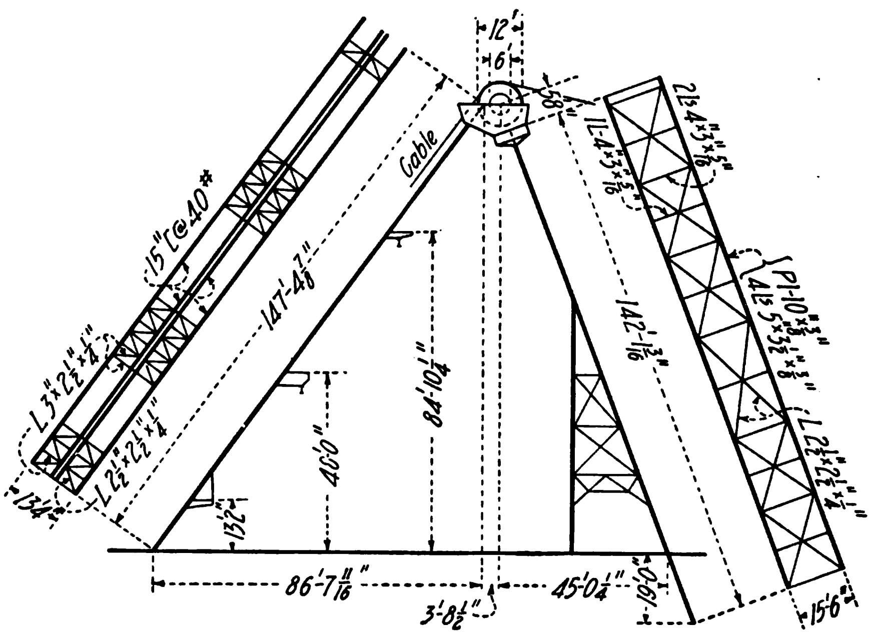

FIG. 108. STEEL HEAD FRAME FOR ROCK HOUSE FOR QUINCY NO. 2 MINE.

7 ft. of the bin is made of No. 10 sheet steel. The crusher floor is supported on four columns, extending up through the bin. These columns have an I-section, and are made of four angles, $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ and a plate 12 in. $\times \frac{3}{8}$ in. These columns have a timber protection, and have braces running out to the bin walls where they are attached to vertical and horizontal angles in the bin walls. Hopper bin *B* is 14 ft. in diameter, and is made of plates $\frac{5}{16}$ in. thick. Poor rock bin *C* is 13 ft. in diameter and 12 ft. 6 in. high, and is made of plates $\frac{5}{16}$ in. thick. Crusher rock bin *D* is 9 ft. in diameter and 9 ft. high, and is made of plates $\frac{5}{16}$ in. thick. Mass copper tube *G* is 5 ft. in diameter, and is

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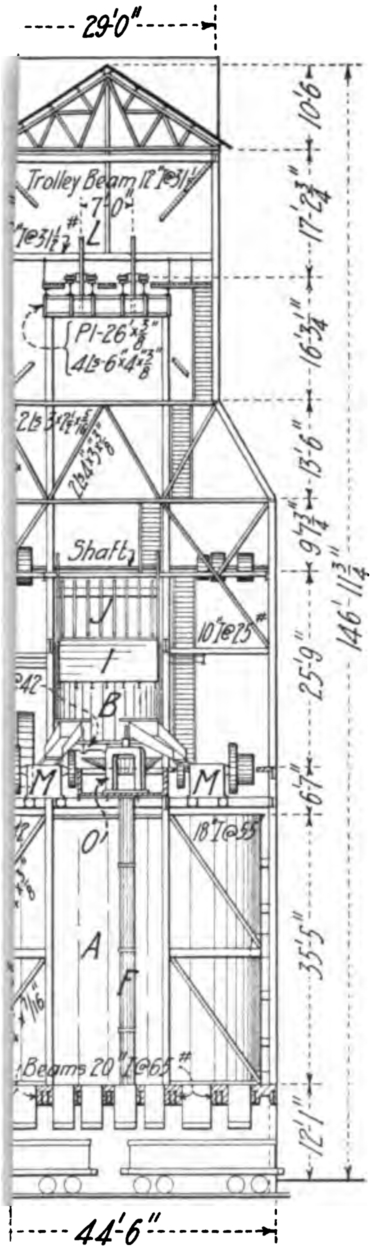

…NSVERSE SECTION

A Stamp Rock Bin
B Hopper Bin
C Poor Rock Bin
D Crushed Rock Bin
E Poor Rock Tube
F Steam Hammer Tube
G Mass Copper Tube
H Big Rock Bin
I Big Rock Chute
J Big Rock Chute
K Poor Rock Chute
L Sheaves
M 40 Ton Crusher
N 20 Ton Steam Engine
O Steam Hammer

STEEL ROCK HOUSE No 2
QUINCY MINING COMPANY
HANCOCK, MICH.
BUILT BY
AMERICAN BRIDGE CO.

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made of plates $\frac{5}{16}$ in. thick. The steel chutes are made of $\frac{3}{8}$ in. steel plate.

The entire building is covered with No. 22 galvanized corrugated steel on the sides, and No. 20 galvanized steel on the roof. The corrugated steel is laid with 4 in. end laps on the sides, and 6 in. end laps on the roof. The purlins are 5 in. channels @ $6\frac{1}{2}$ lbs., and have $\frac{5}{8}$ in. sag rods. The girts are 4 in. channels @ $5\frac{1}{4}$ lbs.

The sheave girders were designed for the breaking strength of the $1\frac{1}{2}$ in. hoisting rope.

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CHAPTER VII.

THE DESIGN OF COAL TIPPLES.

Introduction.—The design of a coal tipple depends upon the quality of the coal, upon whether the coal is hoisted from the shaft or is taken from a drift or tunnel, and upon the work that it is necessary to do in order to prepare the coal for the market. The coal tipple for a bituminous mine in which the coal is hoisted from a shaft, consists of a head frame and a shaker structure or tipple proper where the coal is weighed and screened. A coal tipple for an anthracite mine ordinarily consists of a head frame with storage bins into which the coal is run without crushing or screening; the coal being prepared for market in a separate breaker building. Where bituminous coal is dirty or contains a large amount of refuse material it is sometimes cleaned in a washer building, or is broken, sized and cleaned in a coal breaker. The design of coal washers is considered in Chapter XI, and the design of coal breakers in Chapter XII. (A most interesting and valuable treatise on coal mining methods and appliances is the special report on "Coal Mining," A. C., Pennsylvania Second Geological Survey, 1883.)

With a double compartment shaft the shaking structure, or tipple proper, is usually placed with its axis at right angles to the center line of the two compartments. The hoisting ropes may be either parallel to the axis of the tipple, as in Fig. 12 or Fig. 123, in which case the head sheaves are parallel; or may be placed at right angles to the axis of the tipple, in which case the sheaves are placed in tandem as in Fig. 127 and Fig. 130. The coal may be run through rotary screens, or over shaking screens as is now the common practice. Shaking screens are usually divided into sections and are driven by eccentrics placed 180 degrees apart. The shaking screens do not ordinarily weigh more than two to three tons empty or four to six tons when loaded, but are driven with a velocity of 100 to 150 strokes per minute, with a length of stroke of from 4 to 12 inches, and the shaking motion makes it necessary to design the shaker structure with great care in order to reduce the vibration. The best modern practice in the design of coal

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tipples is to make the head frame and the tipple, or shaker structure, entirely separate and independent units.

Sizing Coal.—The object in sizing coal is to separate the dirt and slack from the coal, and to obtain a product that can be burned more advantageously than unsized coal. A compact coal will not admit the air and will burn on the surface, and it is therefore an advantage to have the lumps of approximately equal size. The sizes and names of the different grades of coal differ considerably in different localities. For the common names and sizes of the different grades of coal, see Chapters XI and XII.

TYPES OF COAL TIPPLES.—Coal tipples may be classed under three types, depending upon the manner in which the coal is brought to the tipple: (1) hoisting in cages or skips from vertical or slightly inclined shafts; (2) cage hoisting on an incline either from a shaft, or on a bridge, or from a tunnel; (3) conveyor hoisting either from the mine or from a head bin into which the coal has been dumped from cars or skips.

1. **HOISTING FROM THE MINE IN CAGES OR SKIPS.**—Bituminous coal is ordinarily hoisted in cars having a capacity of 1 to 3½ tons each, while anthracite coal is usually hoisted in skips or self-dumping cages. A few bituminous mines where the extra handling of the coal is not detrimental to the coal are now using self-dumping skips. Where self-dumping skips are used the coal is dumped into hopper bins at the bottom of the mine, from which it is discharged into the skip. With wet bituminous coal self-dumping skips do not always dump the coal out thoroughly, thus causing a delay. Self-dumping cages are used in both bituminous mines and in anthracite mines—see the tipples for W. P. Rend Coal Company in Fig. 120, the Franklin County Construction Company in Fig. 123, and the Phillips tipple in Figs. 142 and 143. Detail plans of the self-dumping cages used at the Phillips mine are shown in Fig. 144. With plain cage hoisting with cars the waste rock may be raised and dumped during the time of hoisting coal without causing any delay. It is also possible to continue hoisting when there is a delay in the dumping, or the dumping may be continued for a short time when there is a delay in the hoisting.

Plain cage hoisting may be classed as (*a*) single deck, (*b*) double deck, (*c*) single deck and with two cars side by side, (*d*) single deck with two cars end to end. Single deck cages with one car are com-

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monly used for small mines, while single deck cages with two cars end to end have been used in several of the more recent coal tipples.

Dumping Cars in the Tipple.—When the cages have reached the level of the tipple floor the loaded cars may be handled by four different methods.

(*a*) *Push Back Dumps.*—The loaded pit cars are run off the cage by hand, and are pushed to the tipping point and dumped with a push back tipple or dump. After the car is dumped, it is shoved back on the cage, or in the better types of tipples, the empty car is pushed around the shaft and when the loaded cage reaches the landing the empty car is pushed against the loaded car, shoving it off the cage and replacing it. This method is in use in many of the tipples in the Middle West. To handle from 1,000 to 1,200 tons of coal per day of eight hours, with cars of one ton capacity, will require from 5 to 8 men in addition to the weighman and the men employed on the railroad cars. Double deck cages for this form of dumping would be of no advantage. An improved push back tipple is shown in Fig. 110.

(*b*) *Cross-over Dumps and a Ramsey Transfer.*—The tipple floor is arranged to give a descending grade by which the loaded car leaves the cage. With two cars placed end to end, when the cage reaches the level of the tipple floor, a steam or a pneumatic cylinder pushes two empty cars on the cage and pushes off the two loaded cars. Each loaded car then runs by gravity to a cross-over tipple, where, after dumping its load, it is righted, and the next car following automatically opens the locking horns long enough to let the now empty car through. The horns then spring back and catch the incoming loaded car. There are several styles of cross-over tipple, the Phillip's cross-over tipple being shown in Fig. 112, and the Jeffrey-Griffith cross-over tipple in Fig. 113. The empty car then runs down a short incline through a spring switch, and up a short steep incline, which reverses the direction and turns the car into a track which has a down grade to a point back of the shaft. The empty car is here run upon a Ramsey transfer platform. The transfer platform carries two empty cars sidewise up an incline, bringing them in line with one compartment of the shaft. When the cage containing two loaded cars reaches the level of the tipple floor, the empty cars are pushed on the cage by means of the steam or pneumatic ram referred to above, and the two loaded cars are pushed off. This is the method used in the Spring Valley Coal Company's Tipple No. 5, and is shown in detail in Fig. 131 to Fig. 133,

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and in the tipple for the Alberta Railway and Irrigation Company, shown in Fig. 136.

(*c*) *Cross-over Dumps and Chain Haul.*—This method is the same as that described in (*b*), except that a chain car haul is used to haul the car up a steep grade to the level of the cage, and a Ramsey transfer is not needed. This is the plan used for Cardiff No. 2 Tipple, and described in Fig. 127 and Fig 128.

(*d*) *Rotary Dumps.*—Rotary dumps, Fig. 114, may be used in place of the cross-over tipples in method (*b*) or method (*c*).

MINE CAR TIPPLES OR DUMPS.—The coal may be dumped from the mine cars (*a*) by push back tipples, (*b*) by cross-over tipples, or (*c*) by rotary dumps or tipples.

(*a*) **Push Back Dump.**—With a push back tipple the car is run to the tipple horns and is dumped, after which the car is pushed back to a switch and another car is brought up and dumped. This method is slow and expensive of labor. The Jeffrey Manufacturing Company has developed an improved push back dump, Figs. 110 a and 110 b, that is a great improvement on the ordinary push back dump.

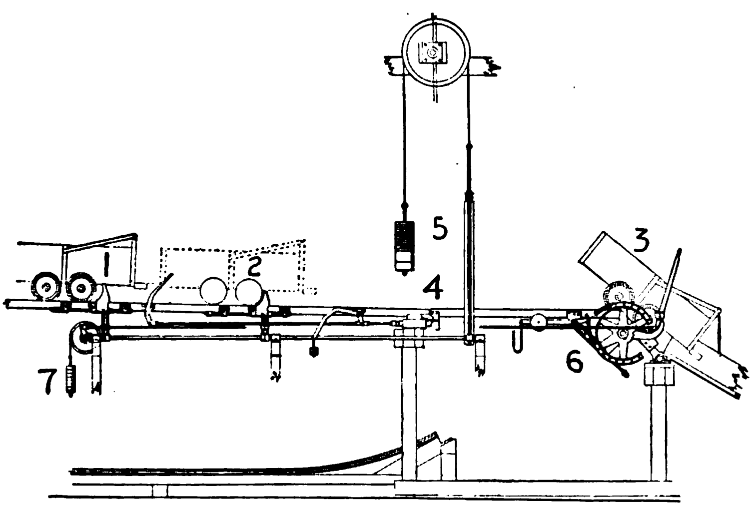

FIG. 110 a. JEFFREY GRAVITY RETURN CAR DUMP.

Jeffrey Gravity Return Car Dump.—The car takes the positions 1, 2, 3 continuously, and is dumped by operating the lever, Fig. 110 a. When the car is righted, the second car moves forward and releases

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the hinged track, and the car 3 passes automatically down the inclined track, Fig. 110 b. After car 3 has passed over the hinged inclined track, it resumes its normal position, and the next car is automatically released and passes to the dumping position. A sketch of a coal tipple in which a return car dump is used is shown in Fig. 111.

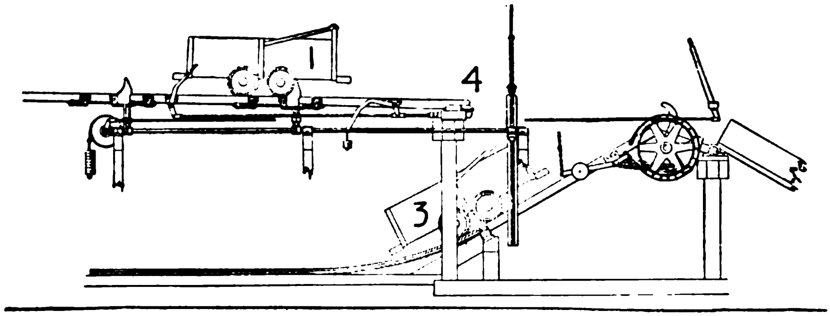

FIG. 110 b. JEFFREY GRAVITY RETURN CAR DUMP.

(*b*) **Cross-over Tipples or Dumps.**—The best known cross-over tipples are the Phillips, the Jeffrey-Griffith, the Mitchell, and the Wilson. The operation of cross-over tipples will be made clear from the description of the Phillips and of the Jeffrey-Griffith tipples.

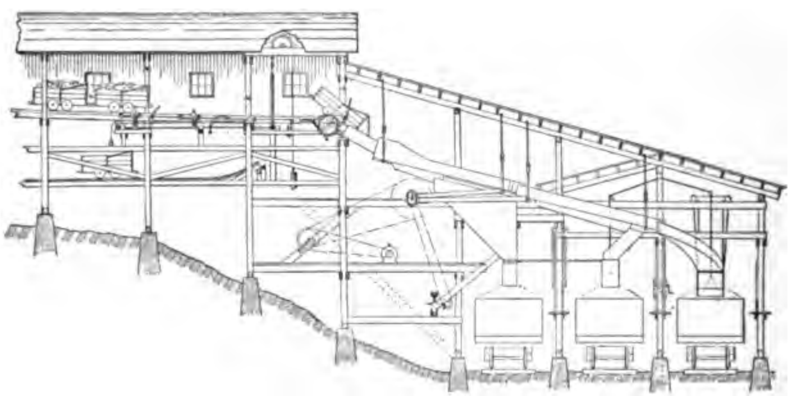

FIG. 111. COAL TIPPLE WITH JEFFREY GRAVITY RETURN CAR DUMP.

Phillips Cross-over Tipple or Dump.—In the Phillips automatic mine car tipple, shown in Fig. 112, hand labor is entirely dispensed with in dumping cars, all the work being done by gravity. The loaded car of ore or coal goes down a one per cent grade and stops on the

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tipple when its front wheel strikes the "horns" A. The man operating the brake lever permits the tipple to tip forward on its rockers R, until the coal or ore slides out of the car into the bin or weighing box. After the load is discharged, the tipple rights itself automatically, the now empty car being held by horns A. When the next loaded car arrives, its front wheels strike the horn spreader B and pushes it forward and downward, pulling chain S which rotates the horn shafts L, to which the horns are keyed. The empty car then coasts down a 15 per cent grade and up the short raised track to a point P, where it comes to rest for an instant before coasting down to the automatic

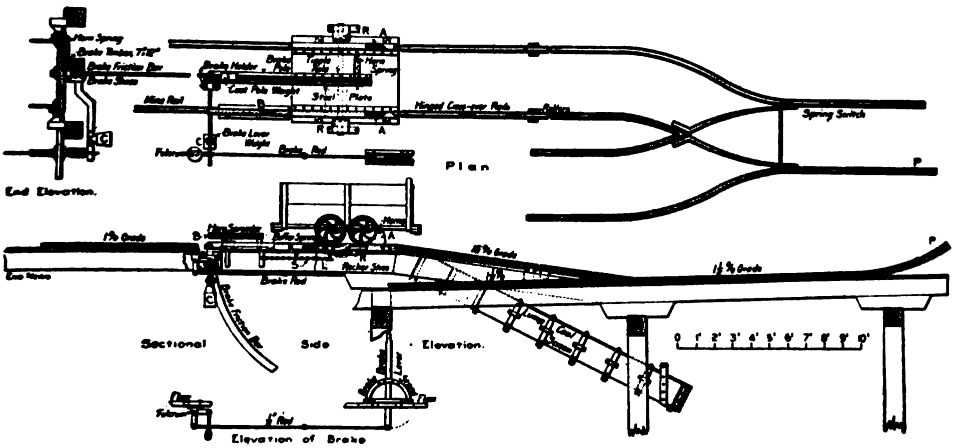

FIG. 112. PHILLIPS CROSS-OVER TIPPLE.

spring switch, and down the track for empty cars. In the meantime the next loaded car has been caught by the "horns" and dumped by the operator. The "horn-shafts" L, on which the "horns" rotate, are provided with heavy buffer springs to take up the jar caused when the loaded car strikes the horns. The "horn springs" pull the cars to an upright position as soon as the rear wheels of the loaded car pass off the "horn-spreader" B. It is therefore impossible for a loaded car to pass the tipple, or to dump itself, for the brake is always locked when the tipple is in its normal position. The tipple weighs from 3,500 to 6,000 pounds, depending upon the size of the car to be handled.

Jeffrey-Griffith Cross-over Tipple or Dump.—The dump consists of two continuous rails to which are attached substantial "horns" securely fastened to a heavy shaft. This shaft has a bell crank and quadrant

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arrangement, with a spring securely fastened to one end, while the releasing and operating lever is at the other end. The car passes on the dump as shown in (*a*), Fig. 113, the jar being taken up by the spring and counterweight, and the car is dumped as shown in (*b*). The car is then released by means of a lever which depresses the "horns,"

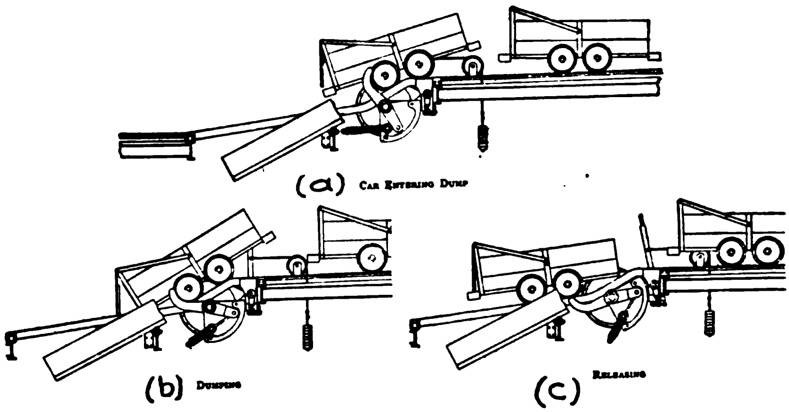

FIG. 113. JEFFREY-GRIFFITH CROSS-OVER DUMP.

and the car moves over the inclined track as in (*c*). The dump then automatically rights itself and is ready for the next car. The manufacturers claim that 10 cars can be discharged per minute. This dump takes up less space than the Phillips dump, and has the advantage of continuous and unbroken rails.

(*c*) **Rotary Dumps or Tipples.**—The car is run into the tipple, Fig. 114, which then makes a full revolution or part of a revolution and

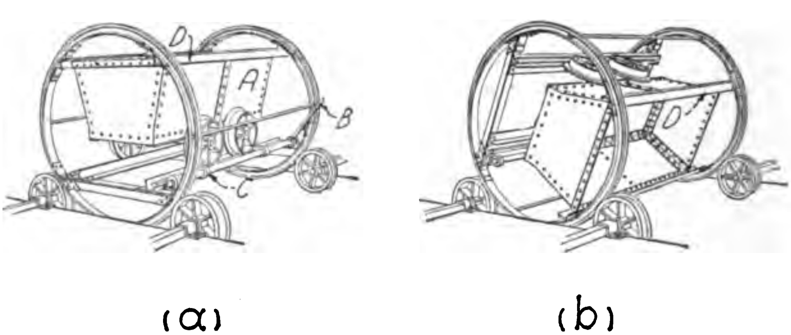

FIG. 114. ROTARY DUMP OR TIPPLE.

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return. Rotary dumps have recently been installed in the steel coal tipple of the Lethbridge Collieries, Ltd., Kipp, Alberta, where the shaker structure is placed at right angles to the tracks on the tipple floor.

2. **CAGE HOISTING ON AN INCLINE OR FROM A TUNNEL.**—At the tipple of the Big Five Coal Company, Fig. 115 a and Fig. 115 b, the coal is delivered at the foot of the incline in cars, where the cars are delivered to a Jeffrey wire cable haul which automatically

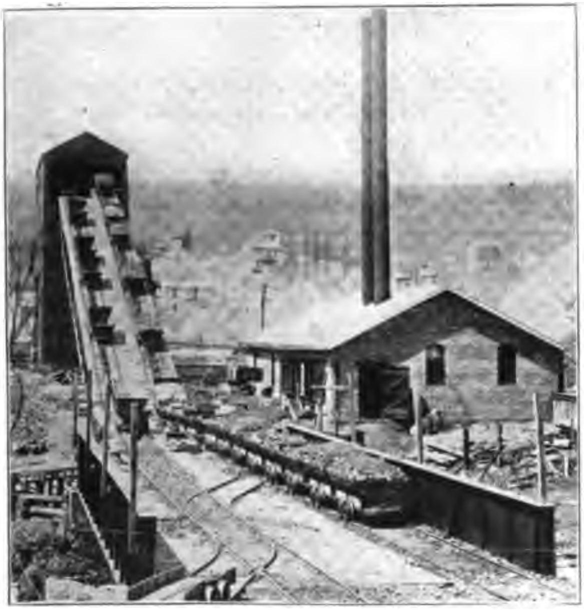

FIG. 115 a. COAL TIPPLE FOR BIG FIVE COAL COMPANY, STEWARTSVILLE, OHIO; BUILT BY JEFFREY MANUFACTURING COMPANY.

picks up the cars and delivers them to the head of the incline, where they are dumped by a Jeffrey-Griffith cross-over dump. The details of the operation of the cars are shown in Fig. 115 a and Fig. 115 b. A tipple in which the cars come in on a level is shown in Fig. 15.

3. **CONVEYOR HOISTING.**—The coal may be hoisted in continuous belt conveyors or in scraper conveyors or bucket conveyors. A coal tipple in which the coal is elevated by continuous conveyors is shown in Fig. 14, while a coal tipple in which the coal is lowered on an incline from the head house bin is shown in Fig. 137 to Fig. 141. The details of the scraper conveyors are shown in Fig. 140 a.

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FIG. 115 b. COAL TIPPLE FOR BIG FIVE COAL COMPANY, STEWARTSVILLE, OHIO; BUILT BY JEFFREY MANUFACTURING COMPANY.

WEIGH BOXES.—An end dump steel weigh box is shown in Fig. 116, and a combined end and center dump weigh box is shown in Fig. 116a.

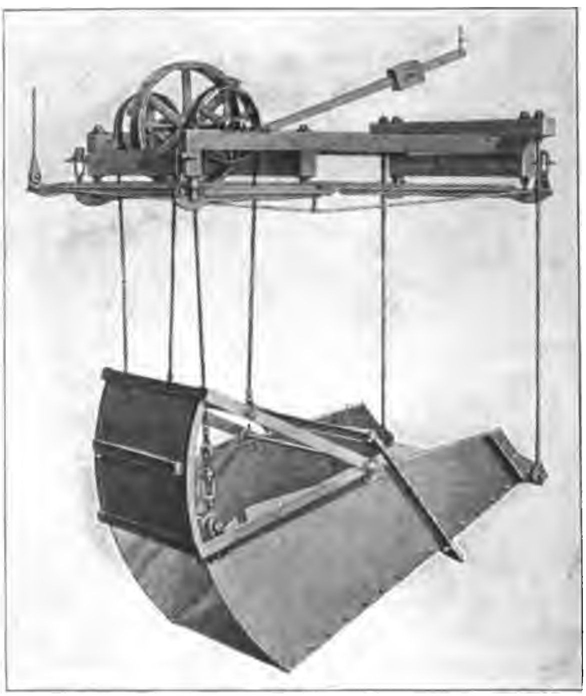

FIG. 116. LINK BELT WEIGH BOX.

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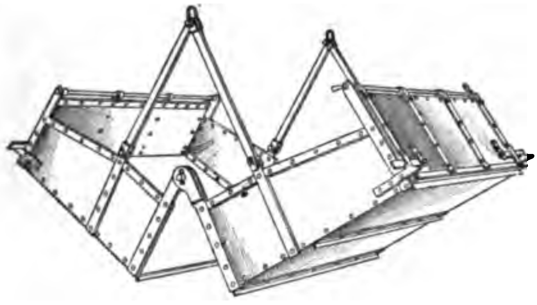

FIG. 116 a. CENTER AND END DUMP WEIGH BOX.

SCREENS.—Screens are of three types: (*a*) rotary or revolving screens, (*b*) shaking screens, (*c*) inclined bar screens. The capacity of screens depends upon the size and slope of the screens, the coal and other conditions. See description of screens in Fig. 121, Fig. 133, Fig. 136, and also Chapters XI and XII.

Circular Screens.—Circular rotating screens have different sizes of mesh so that the different grades of coal varying from lump to slack are obtained from one screen. With fragile coal a rotary screen breaks

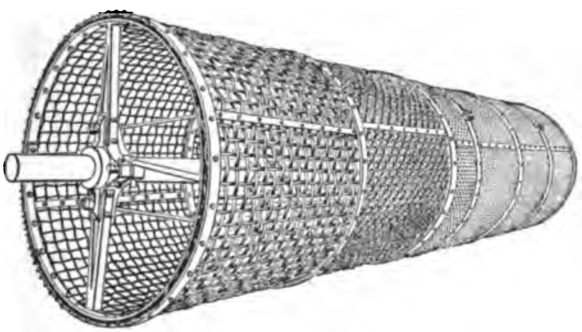

FIG. 116 b. ROTARY SCREEN.

the coal up badly. A rotary screen made of woven wire rods is shown in Fig. 116 b. Rotary screens are also made of perforated steel plates.

Shaking Screens.—Shaking screens are either of the bar type in which the screens consist of rectangular bars spaced at a distance apart as required for the different grades of coal, or perforated screens in which the different size holes are punched in steel plates. In most coal tipples both bar and perforated shaking screens are used. The shaking

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screens are commonly placed at a slope of 3 in. in 12 in., and are either supported by hangers from the framework of the tipple building as in Fig. 111, or the screens are carried on rollers as in Fig. 117a. The

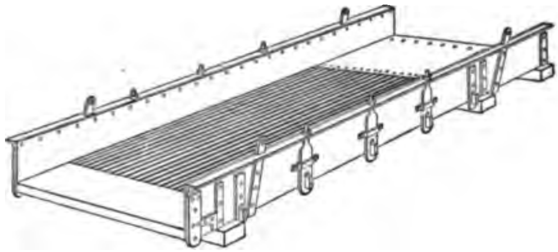

FIG. 116 c. SHAKING BAR SCREEN.

latter method gives much more satisfactory results, as the screens have both a backward and a forward and an up and down motion. Shaking screens are driven by eccentrics attached 180 degrees apart to a shaft

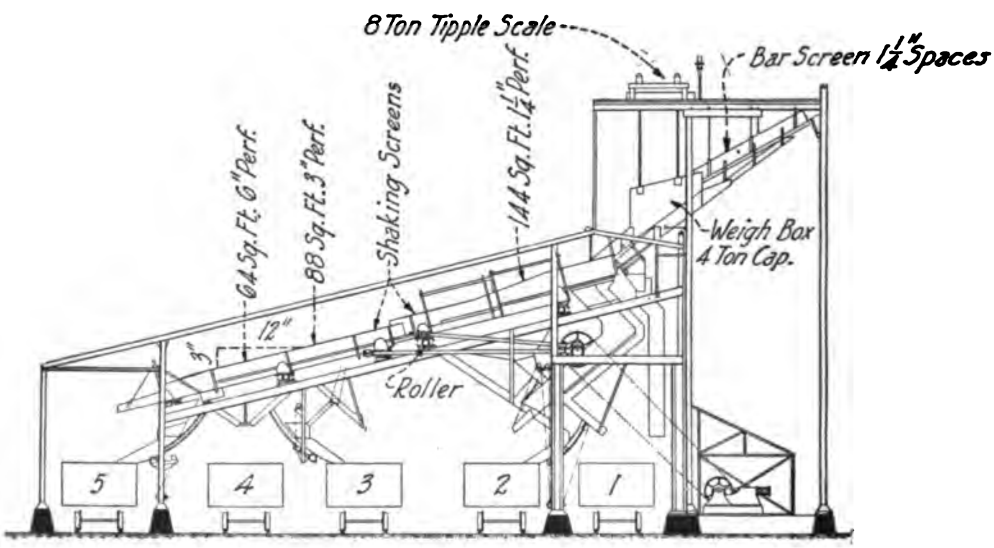

FIG. 117 a. SHAKER STRUCTURE FOR A COAL TIPPLE.

as shown in Fig. 117a and Fig. 121. The stroke of shaking screens varies from 4 to 12 in., and with from 100 to 150 strokes per minute. The plans of a shaker structure for a bituminous coal mine are shown in Fig. 117a. The coal is dumped from mine cars on the bar screens

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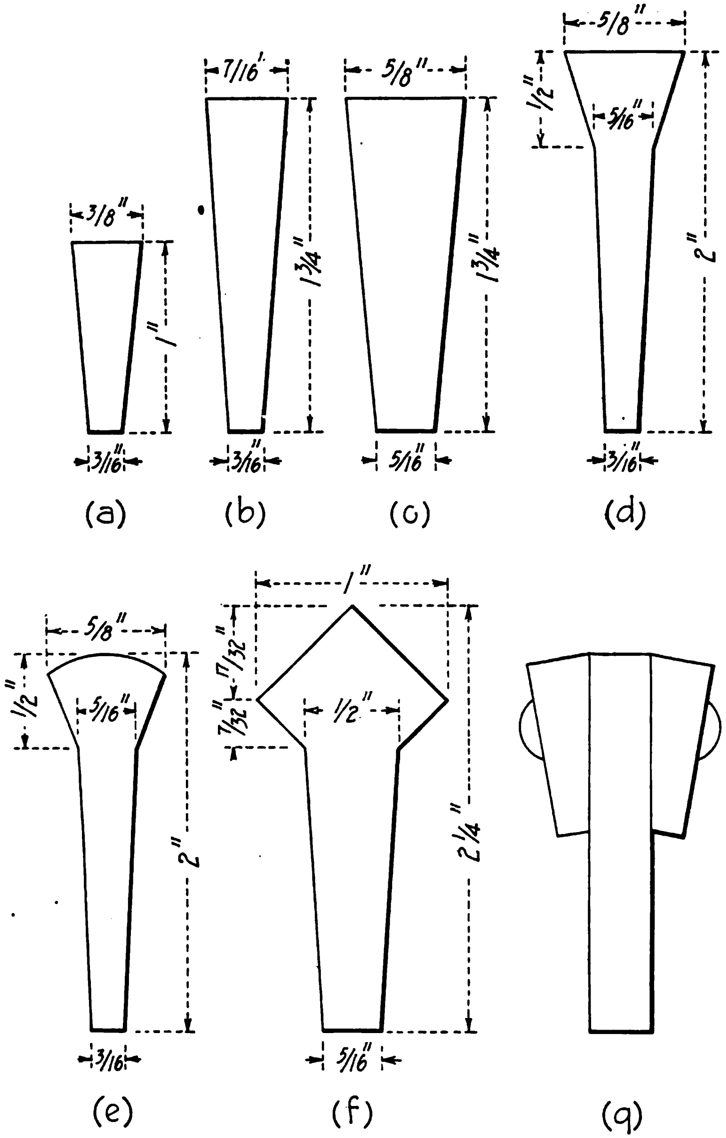

FIG. 117 b. SCREEN BARS.

to remove the slack and dirt from the coal, the coal then runs into a 4-ton weigh box, from which it runs over the shaking screens, or the mine run coal may be run directly into car No. 1. The different sizes

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coal may be obtained by changing the gates in the screens. The
ration of the screens can be seen in Fig. 117 a and in Fig. 121. In
dling soft bituminous coal care must be used to have the spouts
l chutes so arranged that the coal will not be broken up in falling
ough considerable distances. Special loaders are used for loading
l into box cars. The operation of coal tipples is explained in detail
he description of the coal tipple at Shaft No. 5 of the Spring Valley
l Company, the coal tipple of the Alberta Railway and Irrigation
npany, and the coal tipple of the W. P. Rend Coal Company.

Inclined Bar Screens.—The standard sections of flat screen bars
shown in Fig. 117 b. Standard inclined bar screens, over which the
e run coal is dumped, were adopted by the United Mine Workers
1898 as follows:

" Screens hereby adopted for Ohio, Western Pennsylvania, and the
uminous districts of Indiana shall be uniform in size, 6 ft. wide by
ft. long, built of flat acorn-shaped bars of not less than $\frac{5}{8}$ in. surface,
h $1\frac{1}{4}$ in. between bars, free from obstructions; and such screens shall
on a sufficient number of bars to hold the bars in proper position."

PICKING TABLES.—Where the coal in the mine is mixed with
e, rock or other refuse in large pieces it may be necessary to sepa-
the coal and the refuse. Picking tables are wide belts which carry
coal past the pickers, the refuse is picked off the belt and placed
the refuse conveyor and the coal continues on the picking table to
cars. Details of the operation of the picking tables at the coal
ple of the Alberta Railway and Irrigation Company are shown in
. 136; also see Fig. 141.

DESIGN OF STEEL COAL TIPPLES.—The problem of the
ign of the head frame for a coal tipple is the same as for a metal
e, for which see Chapter VI. The shaker structure is a steel frame
lding, for the design of which see Chapter IX. Rigidity is more
ortant than great strength in the design of the head frame and the
ker structure, and care must be used to provide for the loads, and to
perly brace the structure. The head frame and shaker structure
uld be built as independent units. If possible the shaking screens
uld be carried on a framework independent of the shaker building.
the design of bins, see Chapter X. The structural steel in coal tip-
s is subjected to corrosive gases and moisture and the metal should
made thicker than for the usual steel frame structures. The mini-

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mum thickness of steel sections and plates should be $\frac{5}{16}$ in. Allowable stresses and other data for the design of coal tipples are given in "Specifications for Steel Mine Structures," Part II, given in Appendix I; also see "Specifications for Timber Mine Structures" in Appendix II.

DATA FOR COAL TIPPLES.—Data for several coal tipples are given in Table XX; for data on costs of coal tipples, see Chapter XV.

EXAMPLES OF COAL TIPPLES.—Detail plans and descriptions of several coal tipples are given to illustrate the solution of different problems and to explain the principles of design.

Steel Coal Tipple for Hart-Williams Coal Co.—The coal tipple shown in Fig. 118 was built by the Wisconsin Bridge and Iron Co. for

FIG. 118. STEEL COAL TIPPLE FOR HART-WILLIAMS COAL CO.

the Hart-Williams Coal Co., Benton, Illinois. The head frame and shaker building are constructed as independent structures. The shaking screens are set with a skew of 55 degrees with the center line of the head frame in order to accommodate the railroad switch tracks. The sheaves are 8 ft. in diameter, and carry a $1\frac{3}{8}$ in. hoisting cable.

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TABLE XX.

Data on Steel Coal Tipples.

	Depth of Mine, Ft.	Height of Head Frame, Ft. In.	Diameter Sheaves Ft. In.	Size of Hoisting Rope, In.	Method of Hoisting.	Weight of Cage Skip, Lbs.	Weight of Coal Lbs.	Rate of Hoisting. Ft. per Min.	Rate of Hoisting. Tons per Day.	Weight of Structure in Lbs.
Phillips Coal Tipple, Pennsylvania	268	107 0	10 0	$1\frac{3}{8}$	Self dumping cages		4,000		6 tons per min.	569,500
Philadelphia & Reading, Gilberton	1,100	55 0 66 0	14 0	2		40,000 working load each compartment		2,300		
Cardiff No. 2, Cardiff, Ill		65 9 74 3	10 0	$1\frac{3}{8}$	Cars				2,000	180,000
Spring Valley No. 5, Spring Valley, Ill.		75 0 85 6	10 0	$1\frac{3}{8}$					2,000	415,530
Alberta Railway & Irrigation Co., Lethbridge, Alta.	600	83 0 95 0	12 0	$1\frac{1}{2}$	Cars	2,000	8,000		200 tons per hour	500,000
Rend Tipple, Rendville, Ill.		79 6	8 0	$1\frac{3}{8}$	Self dumping cages				2,500	Head Frame 100,000 Shaker 56,000
Carbon Tipple, Carbon, Montana		90 0	9 0		Cars					355,400 Structural steel 16,800 Corrugated steel
R. F. C. Co. Tipple, Montana					Cars					171,200 Structural steel 31,300 Corrugated steel
Gebo Tipple, Montana					Cars					117,200 Structural steel 10,300 Corrugated steel

The capacity of the tipple is 2,000 tons in 9 hours. The head frame was designed for the same loads and unit stresses as the steel tipples built by the same company for the W. P. Rend Coal Company, and the Franklin County Construction Company, which see.

Steel Coal Tipple for Lehigh and Wilkesbarre Coal Company.—The steel head frame built by the Lehigh and Wilkesbarre Coal Company at Sugar Notch, Pennsylvania, is of the 4-post type; the distance from the collar of the shaft to the center of the sheaves being 75 ft., the base being 40 ft. 11¾ in. by 28 ft. 11½ in. The rectangular tower has six posts made of two angles 5″ × 3″ × ⅜″ and is braced with angle bracing. The back brace consists of three columns which are made of two angles 6″ × 6″ × ½″. The main diagonal braces are made of single angles 3″ × 6″ × ⅜″.

The head frame was designed for a live load of 30,000 lbs., acting alternately on each hoist; a dead load equal to the weight of the structure, and a wind pressure of 24 lbs. per sq. ft. of vertical projection. The unit stresses for tension in posts and bracing were taken at 5,000 lbs. per sq. in. for live load, 10,000 lbs. for dead load, 15,000 lbs. for wind load. The extreme fiber stresses in channels was taken at 7,500 lbs. per sq. in.

In this coal tipple a landing platform is placed at a height above the surface so that the cars can be run from the cage directly on the trestle and be delivered by gravity to the coal breaker.

Steel Coal Tipple for Gilberton Shaft.—The Gilberton shaft of the Philadelphia and Reading Coal and Iron Company's mines, Gilberton, Pa., is 1,100 feet in depth and is divided into four compartments. The two compartments nearest the engines are used for hoisting water and are served by the hoisting engines nearest the shaft. The other two compartments are ordinarily used for hoisting coal and rock, although in case of emergency they can also be used for water.

The head frame is of the 4-post type, and is 55 ft. from the top of the shaft to the center of the back sheaves, and 66 ft. to the center of the front sheaves. The center of the rear brace produced passes through the centers of the two pairs of sheaves, which are so located that the line of resultant pressure produced by the load in the shaft and the pull from the engine falls slightly below this plane, thereby avoiding all overturning moment and throwing practically all the pressure produced by hoisting directly on the rear brace. The main columns are made of two channels laced, while the bracing is stiff throughout,

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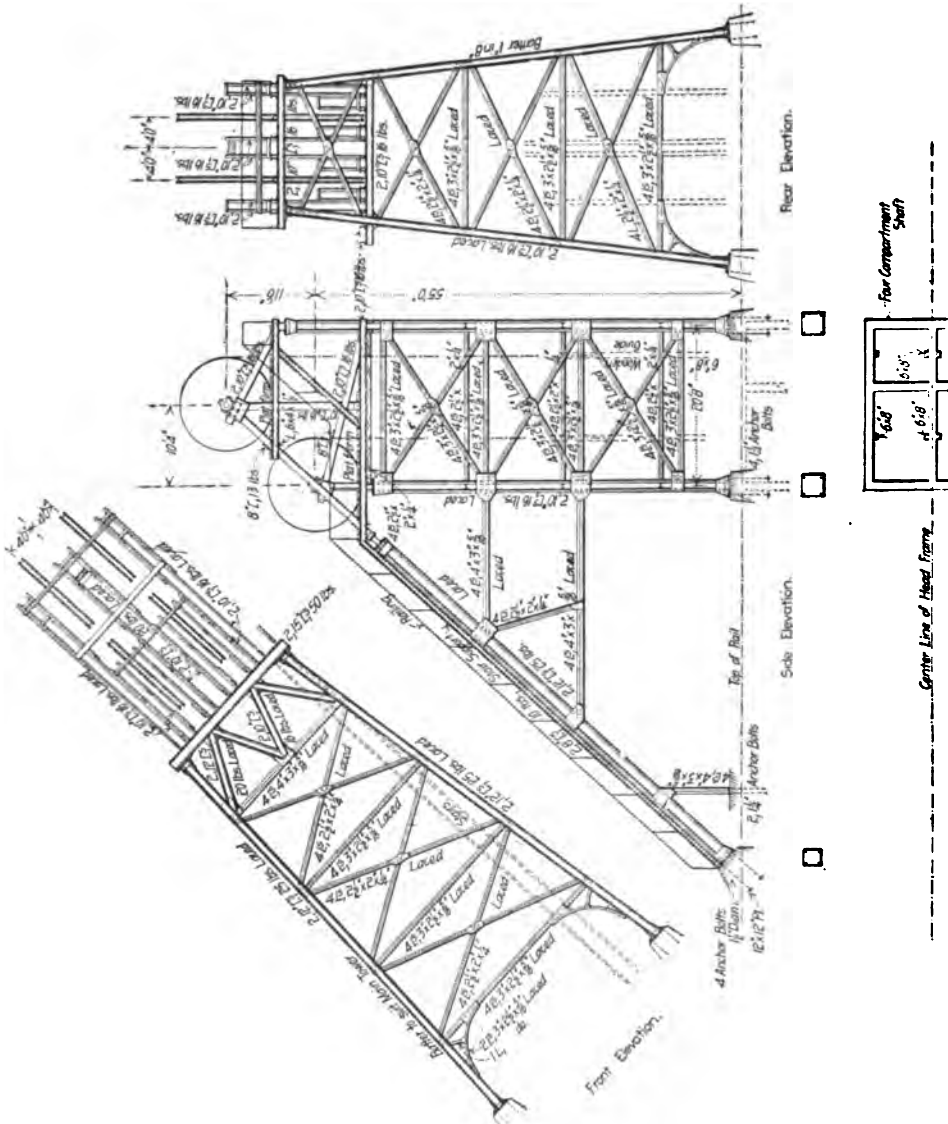

FIG. 110. STEEL HEAD FRAME FOR GILBERTON SHAFT.

consisting of four angles laced in I-section. Detail plans of the head frame are given in Fig. 119.

The sheaves are 14 ft. in diameter with 10 in. journals, the bearings resting on horses inclined to the horizontal, so that the resultant of the live load is nearly normal to them. The hoisting rope is a crucible steel wire rope, 2 in. in diameter. The head frame was designed for a working load of 20 tons in each compartment of the shaft and the engines have a capacity for raising these loads at a speed of 2,300 ft. per minute.

The head frame was designed by W. L. Cowles, M. Am. Soc. C. E., and was erected by the Pottsville Iron and Steel Company in 1898.

Steel Coal Tipple for W. P. Rend Coal Company.—The steel coal tipple for the W. P. Rend Coal Company, Rendville, Ill., has the head frame structure and the shaking structure built independently. The coal is hoisted in self-dumping cages, the tipple being designed to load four tracks, with provision for four extra tracks on the opposite side of the center line of the head frame. The steel head frame is 79 ft. 6 in. from the collar of the shaft to the center of the sheaves. The sheaves are 8 ft. in diameter and carry a 1⅜ in. hoisting cable. A general view of the steel framework of the coal tipple is shown in Fig. 120, while plans of the tipple equipment are shown in Fig. 121. The structural steel framework was furnished by the Wisconsin Bridge & Iron Company, while the tipple equipment was furnished by the Link-Belt Company. The head frame and shaker structures were duplicates of the steel coal tipple for the Franklin County Construction Company, Figs. 122 to 124.

Operation of Coal Tipple.—Detail plans of the shaking screens and tipple equipment are shown in Fig. 121. The coal is raised from the mine in self-dumping cages and is dumped into two weigh hoppers having a capacity of four tons each. From the weigh hoppers the coal passes through a dump chute, and may be run directly into cars on the track or may be run over shaking screens. The first section of the shaking screens is 29 ft. 9 in. long, the top deck, having a length of 16 ft., has 3¼ in. round perforations; the middle, having a length of 18 ft., has 2 in. round perforations, the bottom plate being solid. The upper deck of screens sloping toward the head frame has perforations 3¼ in. to 2 in. round; the second deck has perforations 2 in. to ¾ in. round; the third plate deck has perforations ¾ in. round, the bottom deck being solid. The coal passing over the 2 in. and 3¼ in. round per-

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forations of the main screen may be run back over the shaking screens just described, or may be run over the second shaking screen 27 ft. 4 in.

FIG. 120. W. P. REND COAL TIPPLE; DURING ERECTION.

long and 8 ft. wide. This shaking screen has a length of 8 ft. with perforations 6 in. in diameter. By making different combinations of the screens different grades of coal can be obtained, as is shown in Fig. 121. The shaking screens are carried on rollers 12 in. in diameter, which are operated by eccentric connecting rods with a 12 in. stroke. These rollers give the shaking screens a motion in two directions and give much more satisfactory results than the earlier method of suspending the shaking screens from overhead supports. The capacity of the tipple is 2,500 tons in eight hours.

Steel Coal Tipple, Franklin County Construction Company.—The steel coal tipple shown in Fig. 122 to Fig. 124 was constructed by the Wisconsin Bridge and Iron Company for the Franklin County Construction Company, Benton, Ill. This structure is almost an exact

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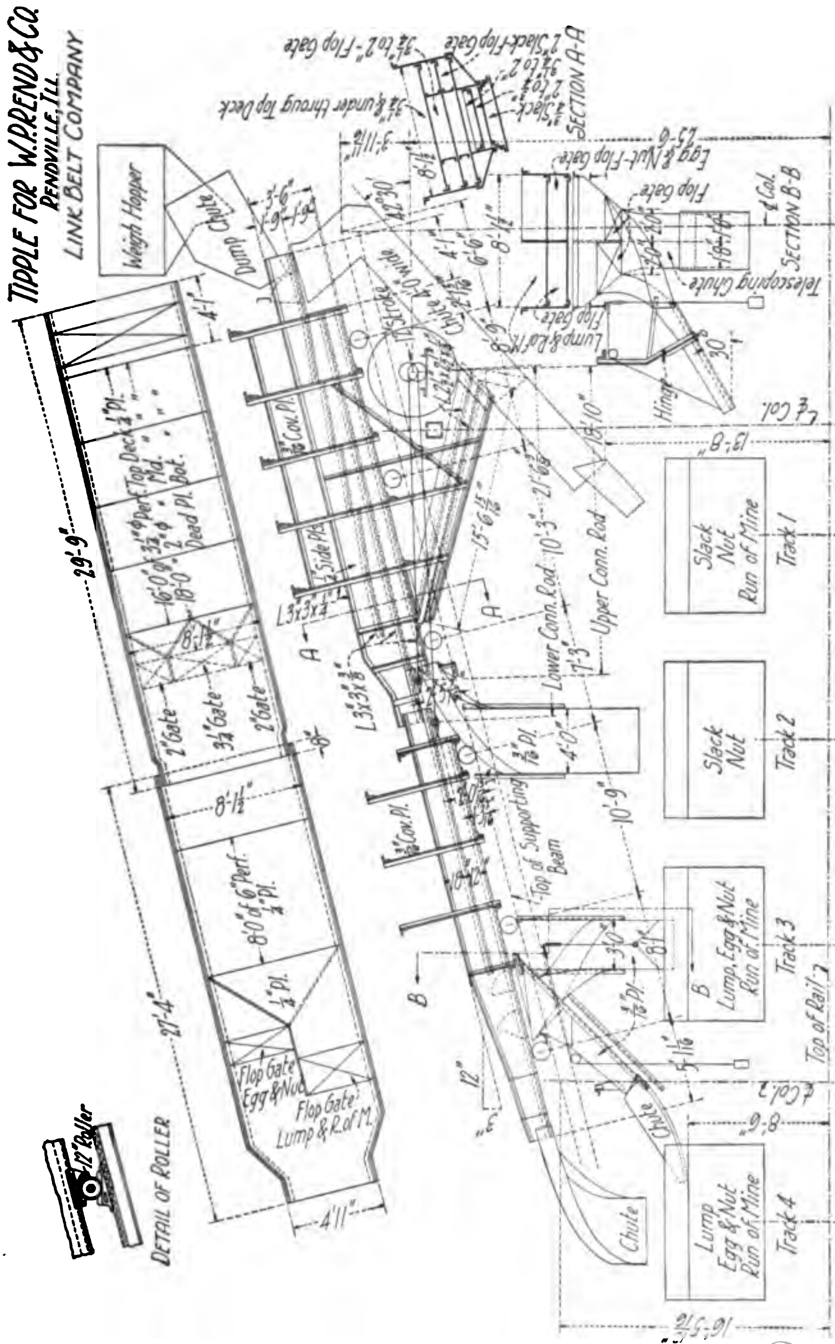

FIG. 121. SHAKING EQUIPMENT FOR W. P. REND COAL TIPPLE.

duplicate of the steel coal tipple erected by the same company for the W. P. Rend Coal Company, Rendville, Ill. The structure consists of a steel head frame and a steel shaker house, the two structures being entirely independent so that the vibrations in one structure will not be transferred to the other. The steel head frame is 79 ft. 6 in. high from the collar of the shaft to the center of the hoisting sheaves. The hoisting sheaves are 8 ft. in diameter and are placed parallel. The hoisting ropes are 1⅝ in. in diameter. The hoisting is done in counterbalance

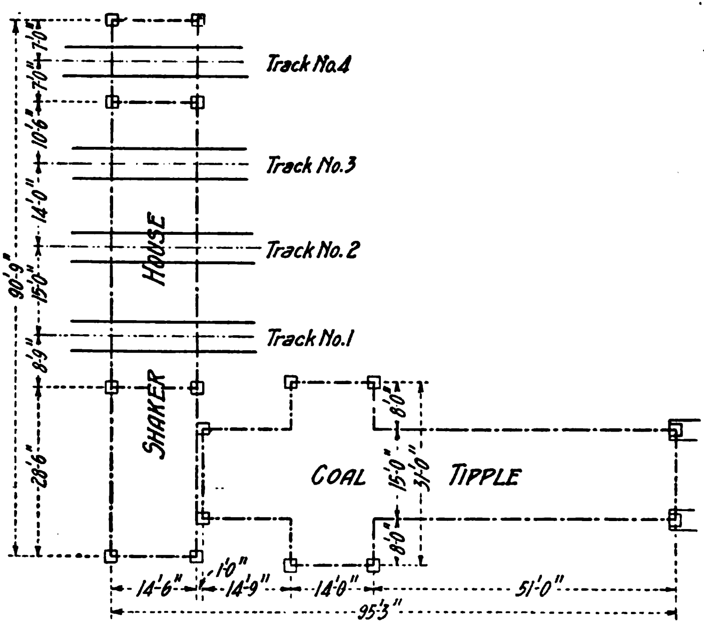

FIG. 122. FOUNDATION PLAN, COAL TIPPLE FOR FRANKLIN COUNTY CONSTRUCTION COMPANY.

with self-dumping cages. The steel head frame is of a modified four-post type, the main sections being single angles. The structure is very fully braced. The weight of structural steel, including the corrugated steel but not including the screens, etc., was for the head frame 100,000 lbs., for the shaker structure 56,000 lbs., total 156,000 lbs.

The shaker house is placed at right angles to the axis of the head frame, as is shown in Fig. 122. The shaker house equipment is practically a duplicate of the shaker house equipment of the W. P. Rend

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Coal Company, shown in Fig. 121, and was furnished by the Link-Belt Company.

The coal tipple was constructed of medium steel under Manufacturer's Standard Specifications. The minimum thickness of the metal

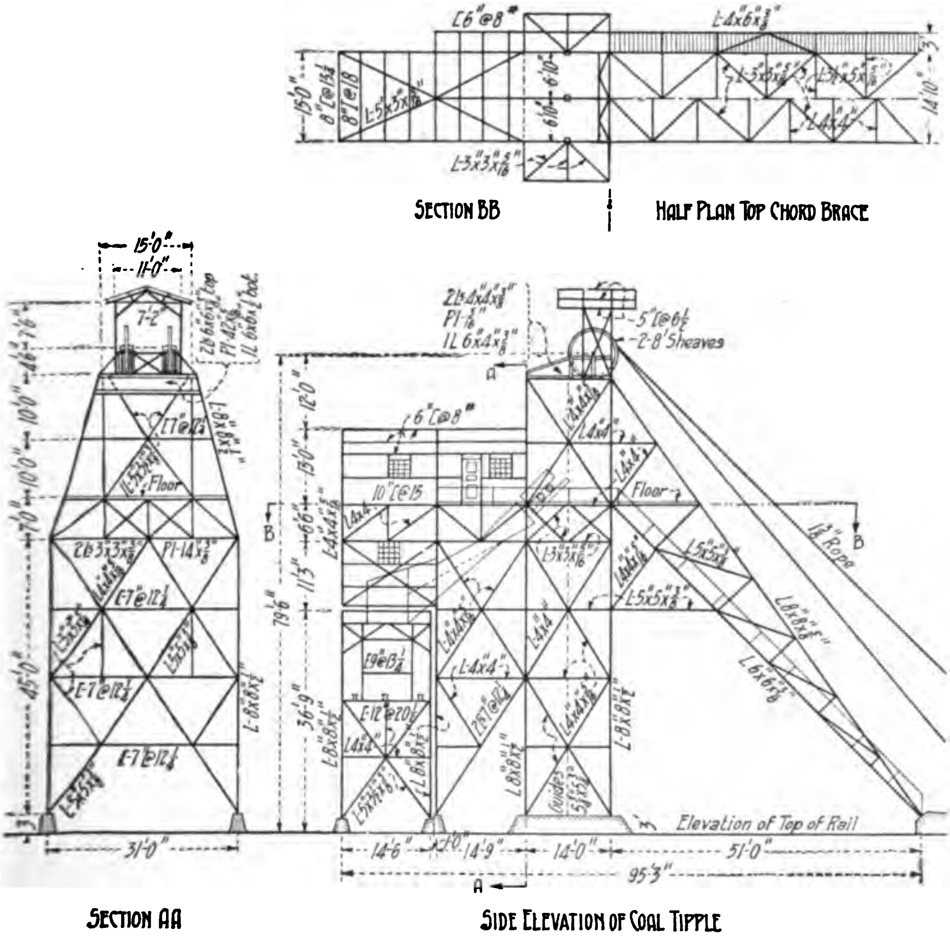

FIG. 123. STEEL COAL TIPPLE, FRANKLIN COUNTY CONSTRUCTION COMPANY.

was $\frac{5}{16}$ in. except for purlins and girts. The connections of the main tower and back bracing and of the shaker structure were riveted, all minor connections being bolted. The tipple was covered with No. 22 gage corrugated steel on the roof and No. 24 gage corrugated steel on

the sides. The windows and doors were made of white pine. The floors and platforms were made of 3 in. yellow pine, nailed to 3 in. spiking pieces.

The coal tipple was designed for a breaking load on one cable of 80 tons and a working load on each cable of 8 tons. The roof was designed for a load of 40 pounds per sq. ft. of horizontal projection.

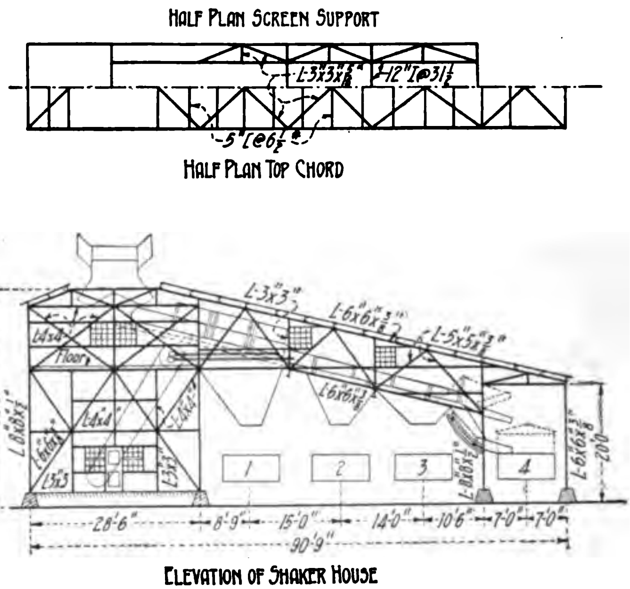

FIG. 124. STEEL SHAKER BUILDING FOR COAL TIPPLE, FRANKLIN COUNTY CONSTRUCTION COMPANY.

The floors were designed for a live load of 150 pounds per sq. ft. For the working load the structure was designed for 15,000 lbs. per sq. in. in tension, and 15,000 lbs. per sq. in. reduced by a standard column formula for compression. For breaking loads the structure was designed for 22,000 lbs. per sq. in. in tension and 22,000 lbs. per sq. in. reduced by a standard column formula for compression.

The Cardiff No. 2 Steel Tipple.—General views of Cardiff No. 2 steel coal tipple, built by the Cardiff Coal Company, are shown in Fig. 125 and Fig. 126; while general detail drawings are shown in Fig. 127. The steel head frame is of the A-type and is built with the hoisting engines at right angles to the main axis of the coal tipple. The legs

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of the head frame are approximately 50 ft. apart at the bottom. The head frame is 65 ft. 9 in. and 74 ft. 3 in. from the top of the shaft to the centers of the sheaves. The bracing consists of struts inclined

FIG. 125. CARDIFF COAL TIPPLE.

FIG. 126. CARDIFF COAL TIPPLE; DURING ERECTION.

from the base to an apex at the center of the platform and again from the edges of the platform to the sheave deck. A second A-frame holds the sheaves. The feet of the tower are securely anchored to concrete

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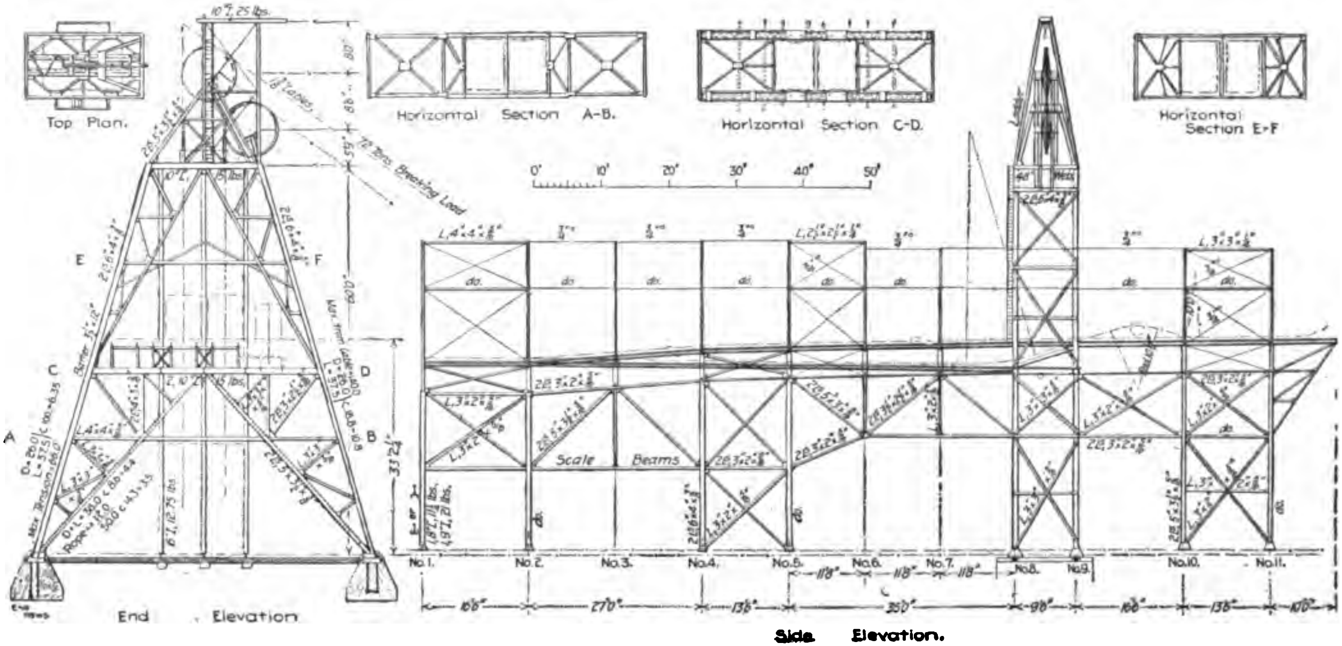

FIG. 127. DETAIL PLANS OF THE CARDIFF COAL TIPPLE.

piers of sufficient weight to take care of any possible uplift, and are connected across by heavy rods imbedded in concrete. The sheaves are 10 ft. in diameter and carry hoisting ropes 1⅜ in. in diameter of extra crucible steel and having a breaking load of 78 tons. The weight of the structural steel, including the corrugated steel, was 180,000 lbs.

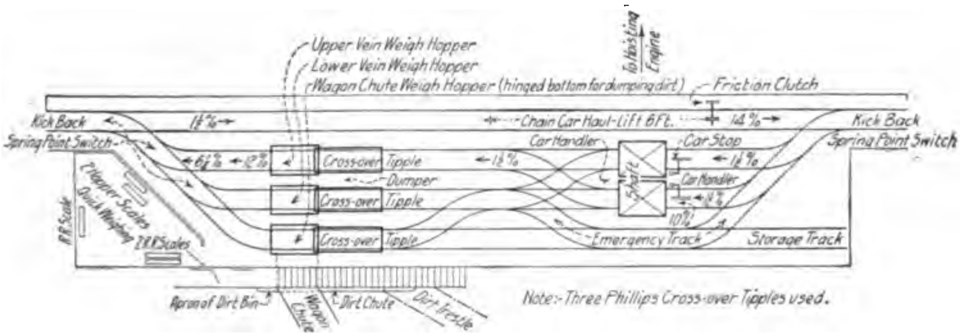

FIG. 128. PLAN OF TIPPLE TRACKS, CARDIFF COAL TIPPLE.

The plans of the tracks in the Cardiff No. 2 tipple are shown in Fig. 128. The coal cars are hoisted on a double deck cage to the floor of the dumping tracks at an elevation of 33 ft. 2½ in. The loaded cars are then shoved off the cage, and may be run over either one of the three Phillips cross-over tipple dumps. The two upper tracks empty into coal hoppers, from which the coal passes to the shaking screens or to the cars for shipment. The lower or left hand track dumps mine run coal or dirt into a chute, as shown in Fig. 128. The empty cars are returned on the upper track by means of a chain car lift and are run on the cage by gravity. The tipple has a capacity of 1,800 to 2,000 tons per day.

Steel Coal Tipple at Spring Valley Shaft No. 5.—The steel coal tipple constructed at Spring Valley shaft No. 5, Spring Valley, Illinois, is one of the best examples of steel tipple construction for bituminous mines. The steel tipple building is 187 ft. long, 36 ft. wide and 35 ft. from the track level to the level part of the main tipple floor. The steel head frame is 75 ft. and 85 ft. 6 in. from the track level to the centers of the sheaves, respectively. The sheaves are 10 ft. in diameter and are placed tandem with the hoisting rope, and at right angles to the axis of the main tipple building. The hoisting rope is crucible steel 1⅜ in. in diameter. The steel tipple building and head frame are cov-

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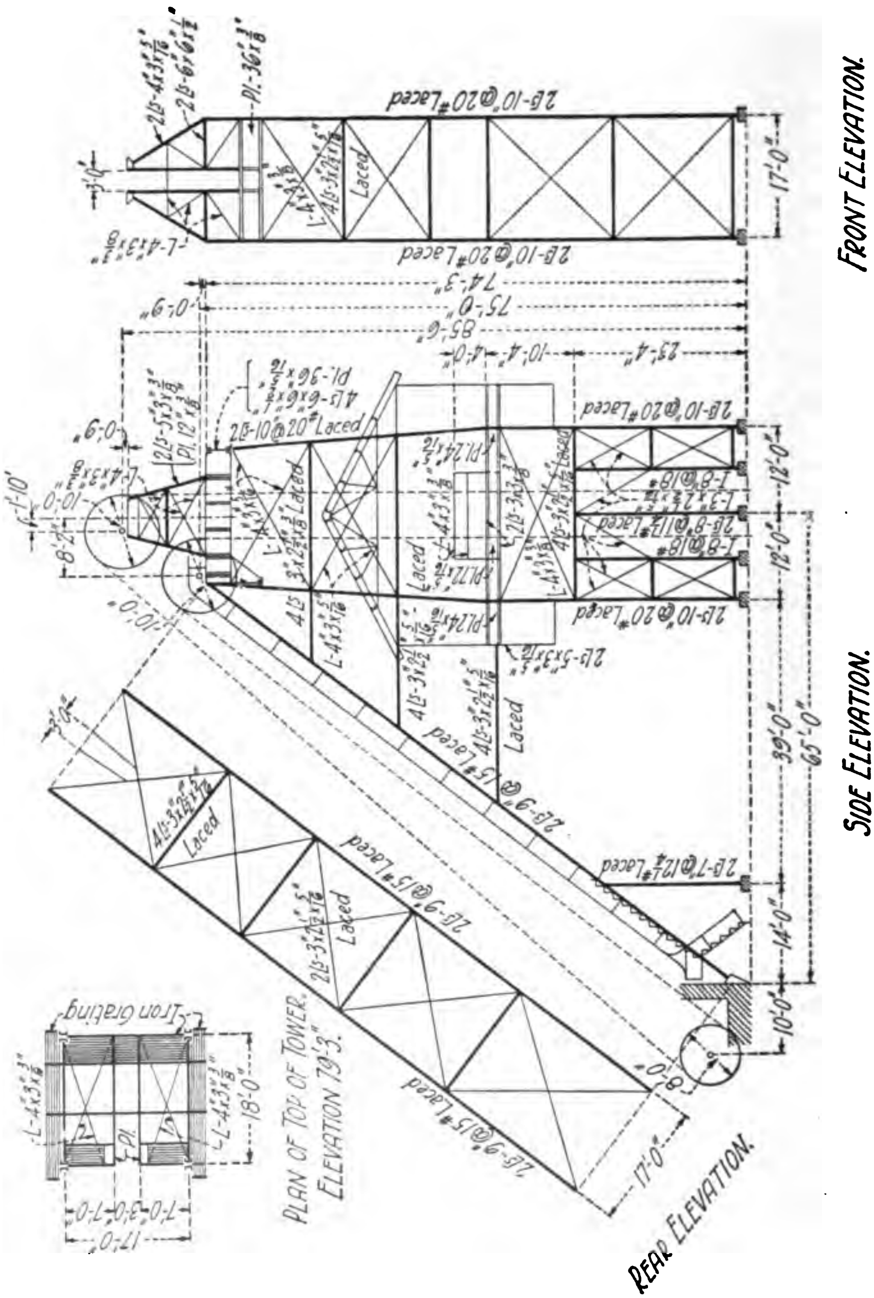
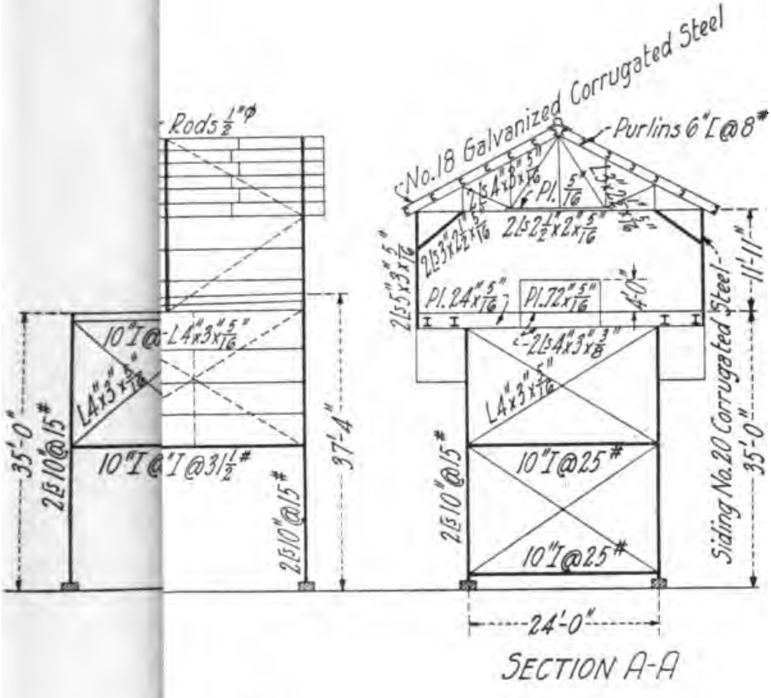
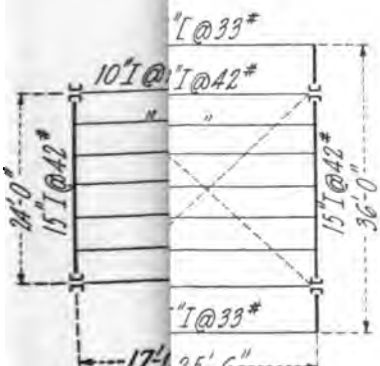


FIG. 130. STEEL HEAD FRAME, SPRING VALLEY COAL TIPPLE, SHAFT NO. 5.





STEEL COAL TIPPLE
FOR
SPRING VALLEY COAL COMPANY'S
SHAFT No 5
SPRING VALLEY, ILL.

Designed by
W. Morava,
Chicago.

Built by
American Bridge Co.

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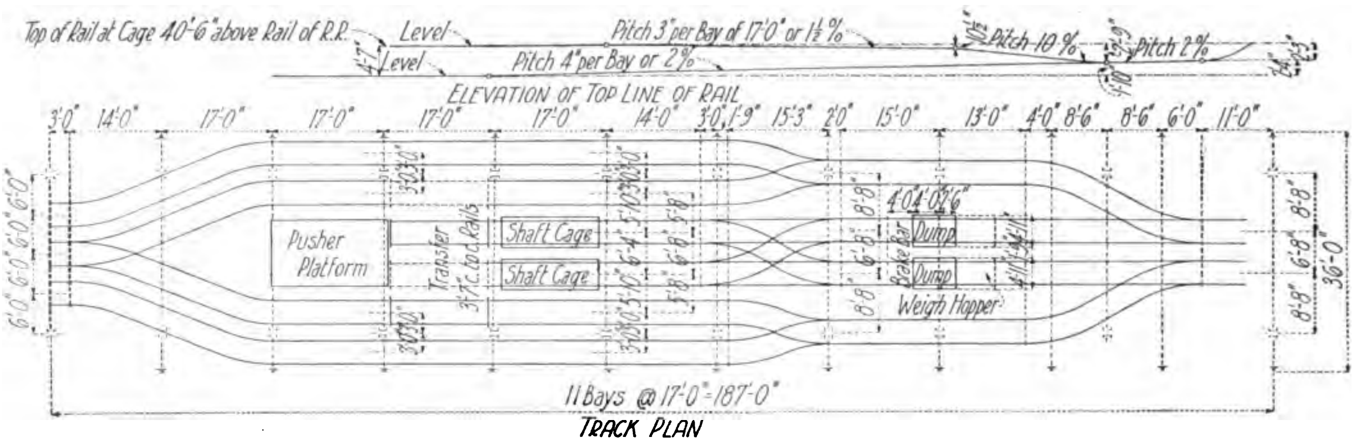

FIG. 131. PLAN OF TIPPLE TRACKS, SPRING VALLEY NO. 5 COAL TIPPLE.

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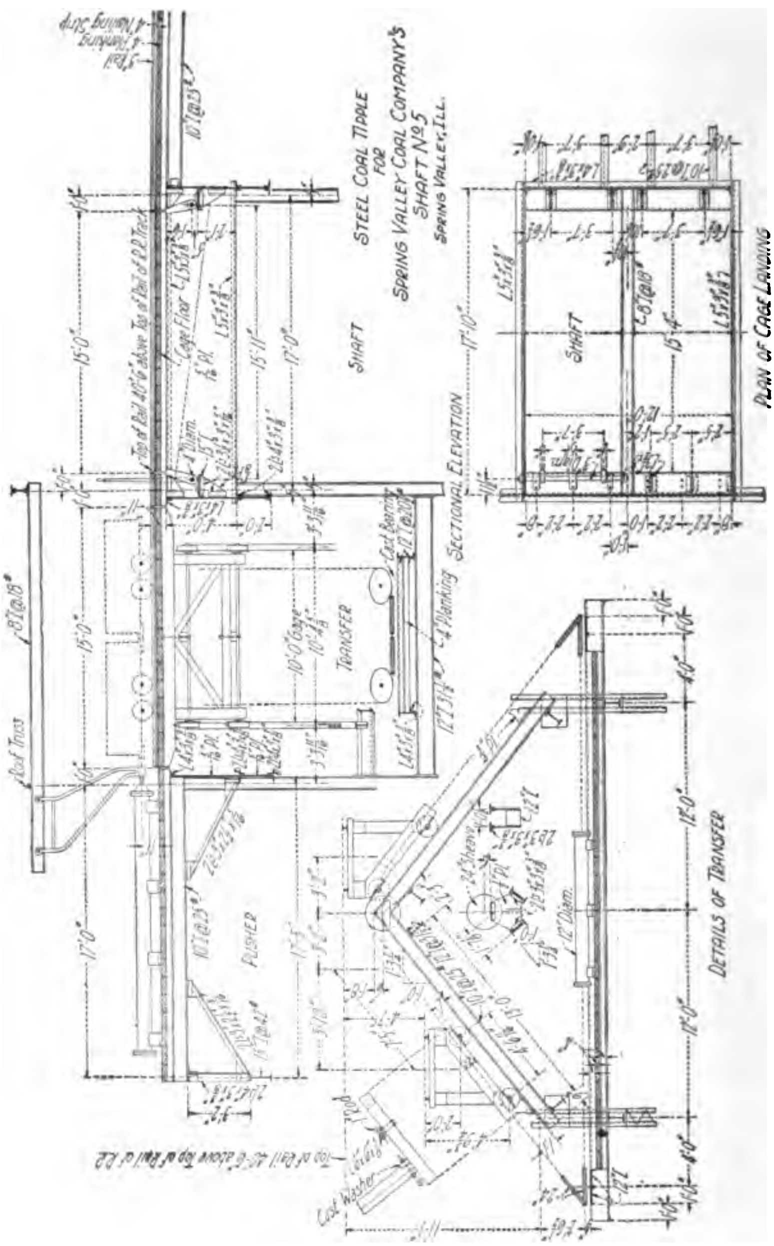

FIG. 132. RAMSEY TRANSFER, SPRING VALLEY NO. 5 COAL TIPPLE.

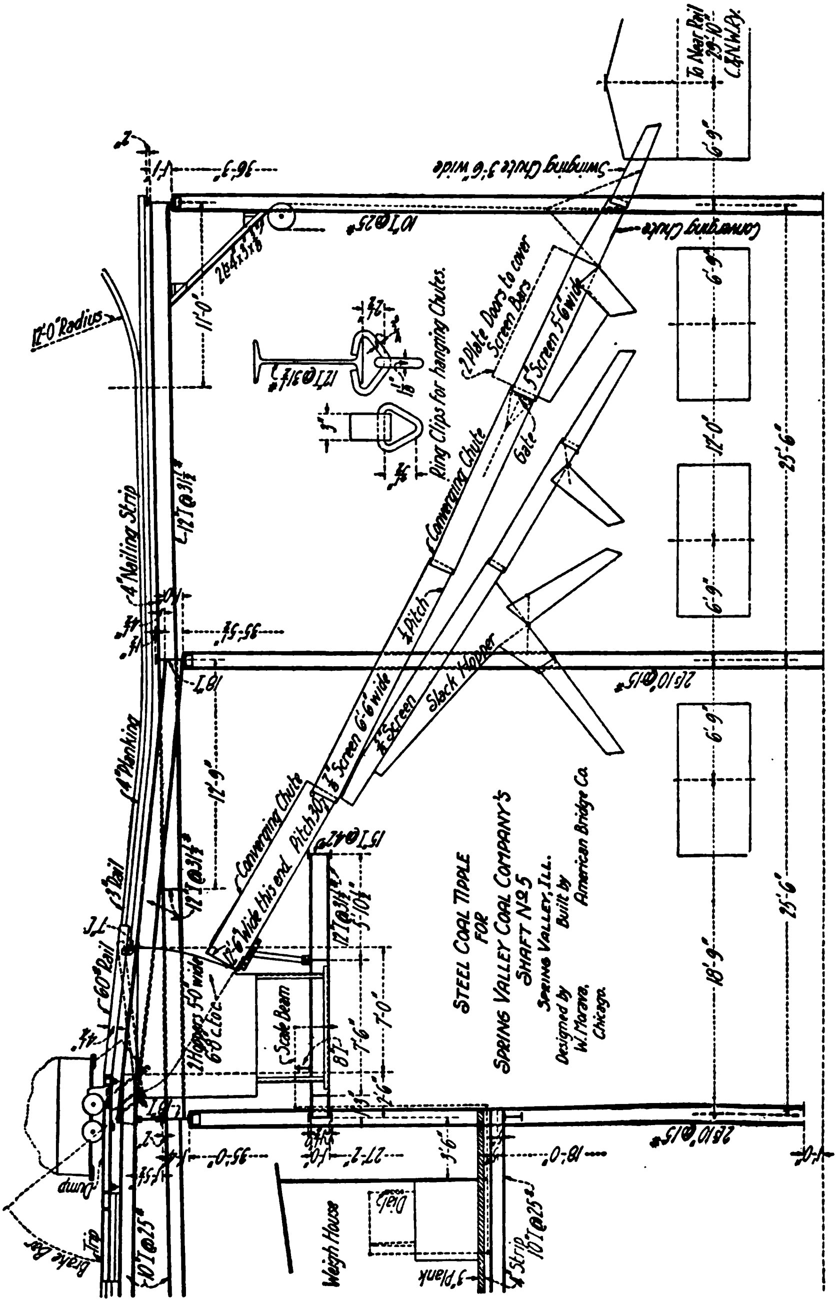

Fig. 133. Shaking Screens, Spring Valley No. 5 Coal Tipple.

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ered with No. 18 galvanized corrugated steel carried on steel purlins. Detail plans of the tipple structure are given in Fig. 129 and of the head frame in Fig. 130. The head frame and tipple building are fully braced and make a very rigid structure. The main track floor of the tipple is level over the first five panels on the left of the structure, the remainder of the floor having a pitch of 4 in. in 17 ft. The tipple floor is covered with 4 in. planking spiked to 4 in. nailing strips which are carried on I-beam joists. The weight of the structural steel, including the corrugated steel but not including tipple equipment, was 415,530 lbs.

Operation of Tipple.—The detail track plan is shown in Fig. 131; the operation of the Ramsey transfer is shown in Fig. 132, and the arrangement of the shaking bar screens is shown in Fig. 133. Two coal cars containing $1\frac{1}{2}$ tons each are hoisted on the shaft cage. The loaded cars are pushed off the cage and two empty cars are pushed on the cage by means of a steam pusher, as shown in Fig. 132. From the cage platform the loaded cars run by gravity on a $1\frac{1}{2}$ per cent grade to the dumps, where the coal is dumped by Phillips automatic tipples or dumps. After dumping, the cars pass to the right by gravity on the 10 per cent descending grade and are stopped by a 2 per cent ascending grade and a short piece of track. The cars then return by gravity, and may either be switched to the outside tracks or run back on the transfer tracks. The empty cars are run on the platform of the Ramsey transfer and are raised by a steam cylinder a height of 4 ft. 7 in. to the level of the floor of the shaft cage, and are ready to be shoved on the cage by the steam pusher.

The coal is dumped by the Phillips tipple dumps into one of two weigh hoppers 5 ft. wide, as shown in Fig. 133. After the coal is weighed it runs out of the weigh hopper on a converging chute having a slope of 30 degrees with the horizontal. From the converging chute the coal runs over shaking bar screens 6 ft. 6 in. wide, the bars being placed $\frac{7}{8}$ in. apart. The fine coal passing through this screen runs over a $\frac{5}{8}$ in. shaking bar screen and is chuted into the cars. The slack passing through the $\frac{5}{8}$ in. bar screen is run directly into the cars. From the $\frac{7}{8}$ in. shaking bar screen the lump coal passes through a converging chute and over a bar screen 5 ft. 6 in. wide with the bars spaced 5 in. apart, from which the lump coal is run into cars. It will be noted that five grades of coal are obtained: mine run coal; lump coal passing over the 5 in. screen; coal passing the 5 in. screen and retained

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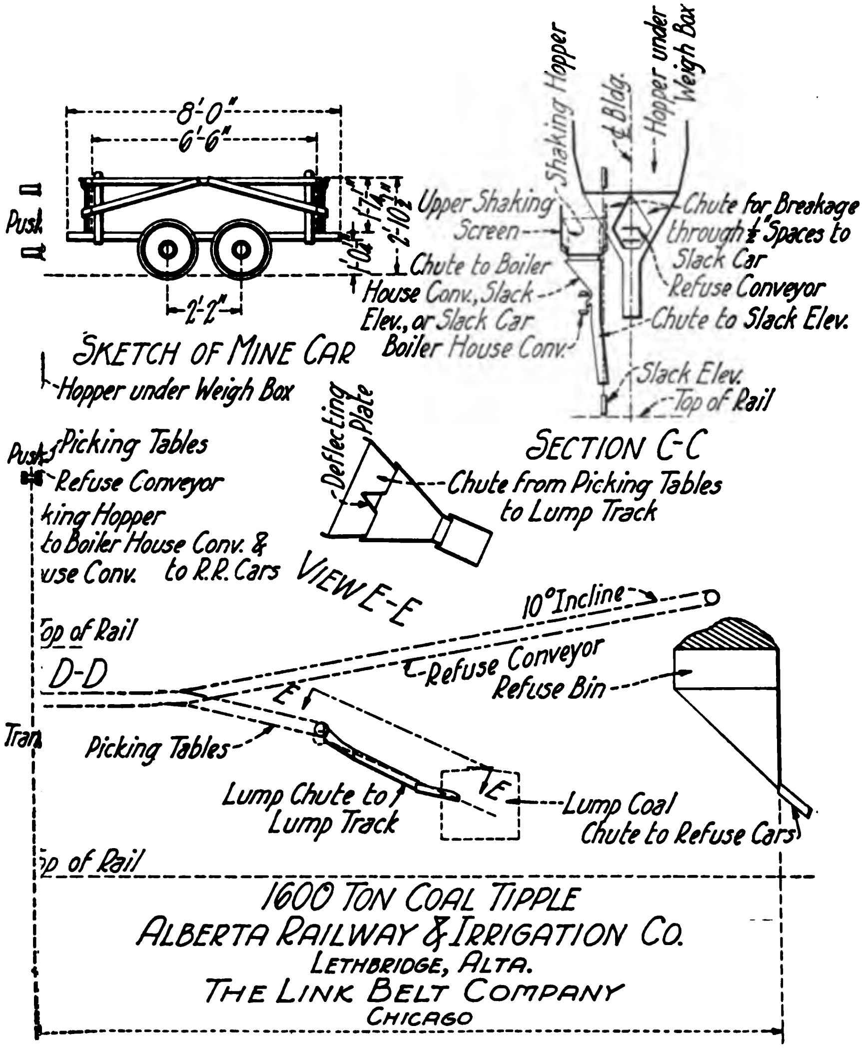

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on a $\frac{7}{8}$ in. screen; nut coal passing a $\frac{7}{8}$ in. screen and retained on a $\frac{5}{8}$ in. screen, and slack.

The capacity of the coal tipple is from 1,800 to 2,000 tons per day. The tipple was designed by Mr. W. Morava, Consulting Engineer, Chicago, Ill., and was built by the American Bridge Company in 1900.

Steel Coal Tipple for the Alberta Railway and Irrigation Company.—The steel coal tipple of the Alberta Railway & Irrigation Company at Lethbridge, Alberta, has a total daily capacity of 1,600 tons of coal. A general view of the coal tipple is shown in Fig. 134, and of

FIG. 134. COAL TIPPLE FOR ALBERTA RAILWAY & IRRIGATION CO.

the steel head frame in Fig. 135. The head frame is of the A-type, and is supported on four columns spaced 24 ft. 7 in. The head frame is 83 ft. high from the collar of the shaft to the center of the lower sheaves, and 95 ft. to the center of the higher sheaves. The sheaves are 12 ft. in diameter and carry hoisting ropes $1\frac{1}{2}$ in. in diameter. The structural steel was furnished by the Wisconsin Bridge & Iron Company. The total weight of structural steel, including corrugated steel, is 500,000 lbs. The operation of the coal tipple is shown in Fig. 136. The details of the equipment were worked out very carefully by the Link-Belt Company, and are worthy of careful study.

Operation of Coal Tipple.—Referring to Fig. 136, it will be seen that the coal is hoisted from the double compartment shaft in cages containing two cars. A sketch of the mine cars is shown in Fig. 136. The cars weigh 2,000 lbs. empty, 4,800 lbs. when loaded with coal, and 6,000 lbs. when loaded with rock. In calculating the capacity of the mine each car was assumed to contain 44 cu. ft. or 1.1 tons of coal. After the cars reach the level of the tipple floor the loaded cars are

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pushed off the cage and the empty cars are pushed on the cage from the transfer truck by means of a pusher cylinder. The loaded car

FIG. 135. COAL TIPPLE FOR ALBERTA RAILWAY & IRRIGATION CO.; DURING ERECTION.

then runs by gravity on a 1½ per cent downward grade to the car dump. After the cars are dumped they continue downward for a distance of 17 ft. on a 12½ per cent grade, then 14 ft. 2 in. on a 2 per cent upward

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grade, and are stopped by a curve with a 16 ft. vertical radius. The cars then return on the outside track by gravity on a 2 per cent downward grade to the transfer track, where they are lifted 5 ft. ½ in. by means of a Ramsey transfer truck. The Ramsey transfer truck is operated by a steam cylinder, as shown in section *A–A*, Fig. 136.

After the coal is dumped it may be run through chutes directly into the rock bin, or may be passed over shaking bar screens. The poor coal or rock is run into a rock bin with a capacity of 1,200 cu. ft. The rock may be drawn from this bin through a 24 in. by 24 in. gate directly into a rock car, from which it is hauled to the dump, or it may be fed through a 24 in. by 24 in. reciprocal feeder with a capacity of 2,000 cu. ft. per hour, on a 24 in. refuse conveyor with a capacity of 50 tons per hour at a speed of 75 ft. per minute.

The good coal is run from the dump over two shaking bar screens 6 ft. by 12 ft. with ¾ in. spaces, having a capacity of 200 tons per hour with 20 per cent passing the two bar screens. The shaking bar screens are set at a slope of 26 degrees with the horizontal. From the two shaking bar screens the coal runs into two weigh boxes with a capacity of 0.88 ton each; the slope of the bottom is 25 degrees before releasing the weigh box and 37 degrees after releasing the weigh box. From the weigh boxes the coal runs into two hoppers, the bottoms of which contain bar screens of 5 ft. × 6 ft. with ½ in. spaces, set at angles of 42 degrees with the horizontal. From the hoppers the coal is fed through two 24 in. × 36 in. reciprocal feeders to the picking table. The two picking tables are made of belts 48 in. wide, and have a capacity of 114 tons each per hour, carrying coal 6 in. deep and run at the rate of 40 ft. per minute. In the design the picking tables were assumed to run seven tenths of the time on account of incidental stops due to shifting cars. The stone and slate from the coal is taken from the picking table and placed on the refuse conveyor which carries it into a refuse bin with a capacity of 1,400 cu. ft. The refuse is drawn from the refuse bin into cars through a spout having a slope of 45 degrees. The lump coal which has passed over the ¾ in. bar screens is run over the picking table and through chutes into cars. Cars may be loaded at the rate of one car in ten minutes, three minutes of which is taken up in shifting the cars.

The slack passing through the ¾ in. bar screens placed above the weigh box runs through spouts on 5 ft. × 14 ft. shaking bar screens with ¼ in. spaces, and then over 5 ft. × 10 ft. shaking bar screens with

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½ in. spaces. Both of these shaking screens have a slope of 3 in. in 12 in., and are driven at a speed of 100 strokes per minute. The slack passing through the ¼ in. bar screen runs through a chute, having a minimum angle with the horizontal of 45 degrees, to a slack elevator inclined at an angle of 60 degrees with horizontal, and having a capacity of 30 tons per hour when running with a speed of 100 ft. per minute. The slack passing through ¼ in. bar screens may also be run directly into the slack car. The slack passing through the ½ in. bar screens, placed in the bottom of the main hopper, is chuted directly into the slack car.

In hoisting, the tipple has an actual storage on the tipple floor of 15½ tons. The rate of hoisting is 200 tons per hour, which requires 3 cars or 3.3 tons per minute. The storage is therefore sufficient for 4⅔ minutes run. Twenty per cent of the coal hoisted goes through the gravity screens, leaving 160 tons per hour or 2.7 tons per minute to go to the lump coal track. At 2.7 tons per minute a 27-ton railroad car can be loaded in ten minutes, assuming that three minutes is taken up in shifting cars, a 27-ton car must actually be loaded in seven minutes. If the dump and picking tables are stopped while the cars are being shifted, their capacity while running is 3.8 tons per minute, or 228 tons per hour.

Steel Coal Tipple for Empire Coal and Coke Company.*—The steel coal tipple built by the Jeffrey Manufacturing Company for the Empire Coal and Coke Company at Landgraff, W. Va., is shown in Fig. 137 to Fig. 141.

Operation of Coal Tipple.—The coal is brought from the mine in mine cars and is dumped by means of two Phillip's cross-over dumps into a receiving hopper in the tipple house having a capacity of 10 tons. From the receiving hopper the coal is fed into a steel plate conveyor by an automatic plate feeder. The conveyor is of the scraper type, and is 129 ft. long with a descending grade of 4½ degrees. The scrapers or flights are 12 in. high by 5 ft. wide, are spaced 3 ft. apart, and are carried by two strands of steel-thimble roller chain. The conveyor trough is made of ⅜ in. flange steel. The conveyor is shown in Fig. 140 a.

The scraper conveyor delivers the coal to the shaking screens, four in number. Three of the screens are in line and pass the coal to the picking tables, while the fourth screen is placed beneath the second and

* Engineering and Mining Journal, August 20, 1910.

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third screens. The first screen is 8 ft. wide by 34 ft. long, with 26 ft. of Jeffrey cross-patent flange lipped screens, which removes the slack and fine coal and passes the lump coal to the picking tables. The second screen is 8 ft. wide by 17 ft. long with 14 ft. of perforations for separating egg coal. Both the first and second screens are fitted with veils for handling mine run coal when the slack is not to be removed. The third screen is a shaking chute transferring the lump coal from the second screen to the picking tables. Screen 3 makes nut coal, which passes to a revolving screen.

Fig. 137. Steel Coal Tipple for Empire Coal and Coke Company.

From the shaking screens the coal is run over the picking tables, Fig. 140 b and Fig. 141, where the slate, sulphur, bone, and other refuse is picked out. The picking tables are constructed as metal apron conveyors, and consist of double beaded flights mounted on two strands of chain operating at a speed of 25 to 50 ft. per minute. The picking tables are provided with adjustable loading booms to prevent breakage. The slack is stored in the slack storage bin, and the pulverized coal is carried to the pulverized coal storage bin, Fig. 139. The pulverized coal is fed directly into coke ovens by larries. The different grades of coal may be mixed automatically if desired. The Pocahontas coal is very fragile and great care is used to prevent breakage.

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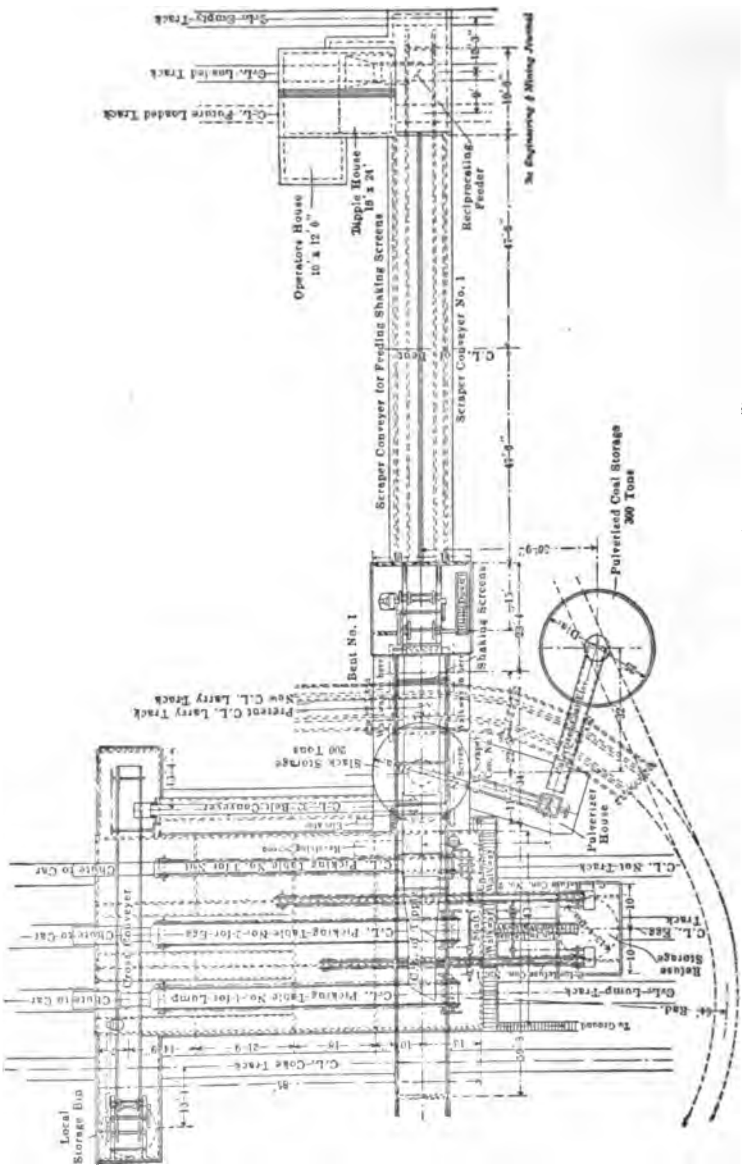

FIG. 138. PLAN OF EMPIRE STEEL COAL TIPPLE.

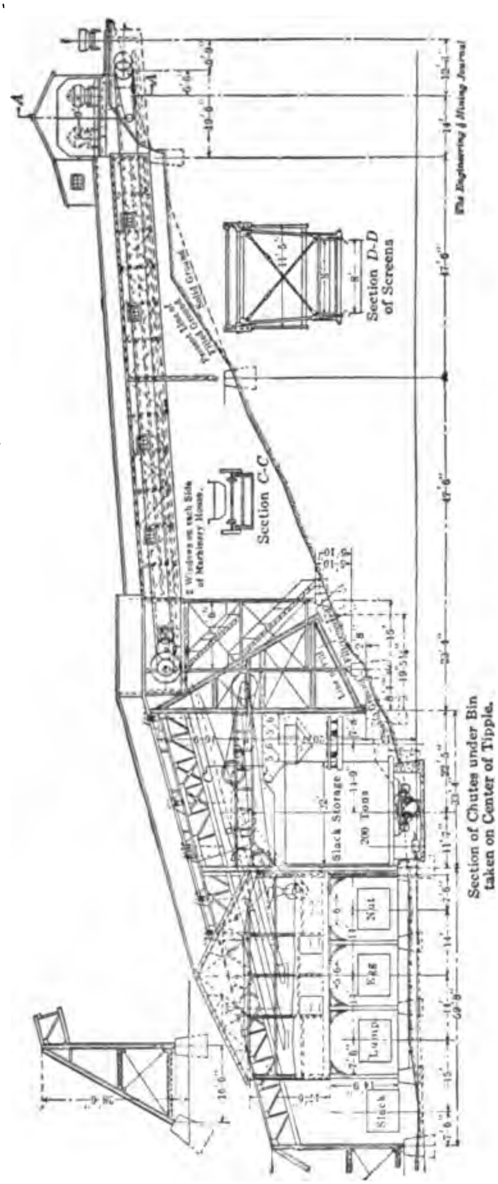

FIG. 139. ELEVATION OF EMPIRE STEEL COAL TIPPLE.

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FIG. 140 a. STEEL SCRAPER CONVEYOR, EMPIRE STEEL COAL TIPPLE

FIG. 140 b. SHAKING SCREENS DISCHARGING ON PICKING TABLES, EMPIRE STEEL COAL TIPPLE.

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The plant was designed to handle 350 tons per hour at a slow speed, or 750 tons per hour at a high speed. The screens can handle 750 tons per hour with 92 revolutions of the eccentric shaft per minute.

FIG. 141. STEEL PICKING TABLES, EMPIRE STEEL COAL TIPPLE.

Design of Coal Tipple.—The entire structure is built of structural steel and is covered with corrugated steel. The shaking screens are supported by a separate structure which does not come in contact with the main building. The shaker house is entirely separate from the remainder of the structure. The shaking screen supports were made very rigid to reduce the vibrations.

The bins are made of steel plates and are lined with a 3 in. layer of Portland cement plaster reinforced with expanded metal. The expanded metal was wired to metal studs which were fastened to the bin plates.

Steel Coal Tipple for the Phillips Mine.—The steel coal tipple at the Phillips mine of the H. C. Frick Coke Company is an excellent example of a modern coal tipple for handling bituminous coal. A general view is shown in Fig. 142 and detail plans of the coal tipple are shown in Fig. 143. The steel head frame is of the 4-post type, and is 107 ft. from the collar of the shaft to the center of the sheaves. The

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main tower of the head frame has six posts made of 4 Z's $3'' \times 2\frac{11}{16}'' \times \frac{3}{8}''$ with one plate $6'' \times \frac{3}{8}''$. The back braces consist of three columns having the same section as the main posts. The head frame is fully cross-braced with angle struts, as shown in Figs. 142 and 143. The batter of the main tower columns is 1 in. in 12 in., while the back brace

FIG. 142. COAL TIPPLE FOR PHILLIPS MINE; BUILT BY AMERICAN BRIDGE CO.

makes an angle of 30 degrees with the vertical. The sheaves are 10 ft. in diameter and are supported on I-beams, resting at the end nearest the engine house on a built-up frame of angles and plates carried on two 15" I-beams, so as to make the necessary clearance for the sheaves. The roof trusses above the sheaves carry two I-beams, on the lower flanges of which are trolleys arranged for the attachment of chain

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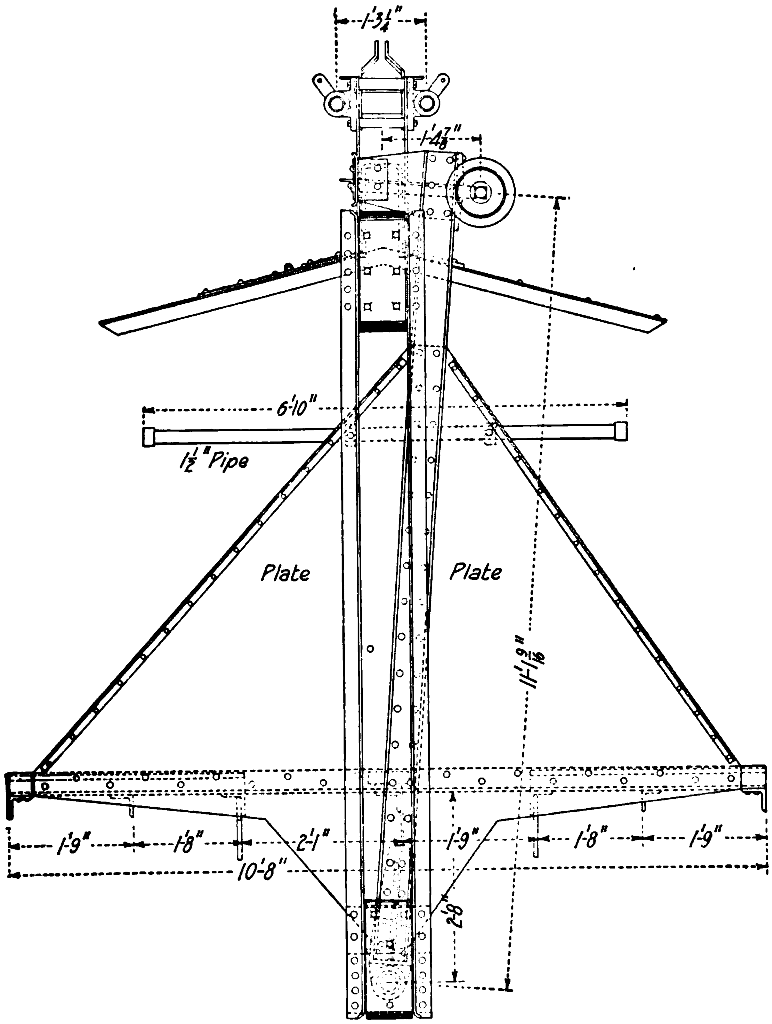

SIDE ELEVATION

FIG. 144 a. DETAILS OF SELF-DUMPING CAGE.

blocks for placing and replacing the sheaves. The shipping weight of structural steel, including the corrugated steel, was 569,500 lbs.

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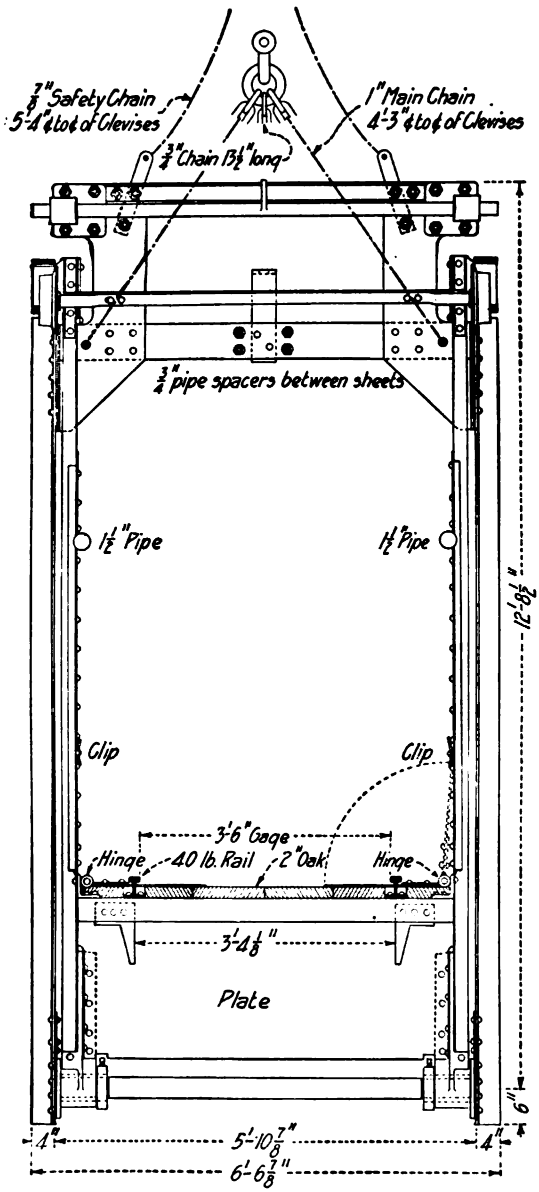

END ELEVATION

FIG. 144 b. DETAILS OF SELF-DUMPING CAGE.

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The coal is hoisted in self-dumping cages which dump the coal into distributing chutes, in which it runs by gravity to the bins having a capacity of 800 tons. The coal, being all used for making coke, is not screened or weighed.

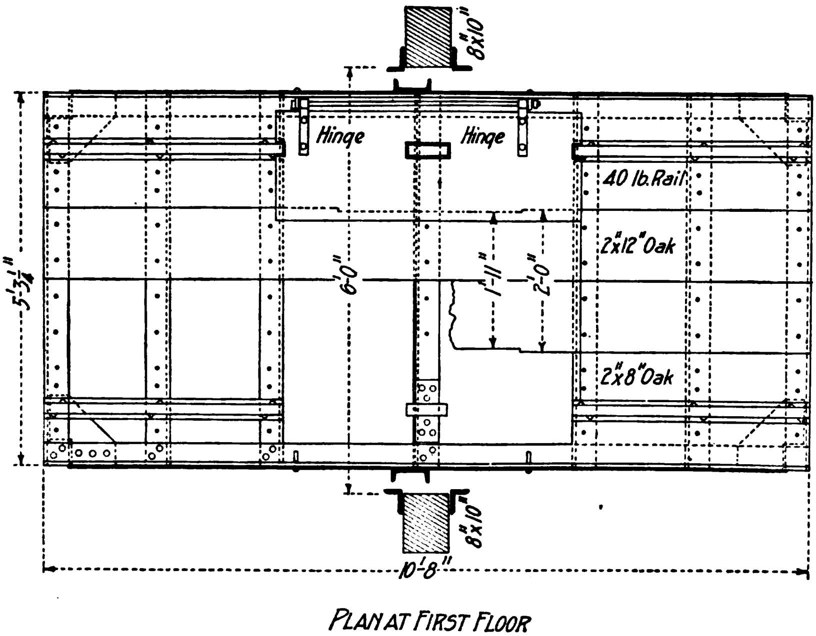

FIG. 144 c. DETAILS OF SELF-DUMPING CAGE.

The storage bins are built with a steel framework and are lined with ¼ in. buckle plates on the sides, and have a ⅜ in. plate floor. The sides are supported by the 15" I-beams @ 42 lbs., spaced 3 ft. 5¼ in. center to center. The inclined bottom framing consists of girders having 48 in. × ⅜ in. web plates and flanges composed of two angles 6" × 6" × $\frac{7}{16}$", and are tied together with ties consisting of two angles 8" × 8" × ¾" and one plate 17 in. × ½ in. at the bottom, and 15" I-beams @ 42 lbs. at the top, the girders being spaced 3 ft. 5¼ in. center to center. The main side girders are composed of two I-beams 15" @ 42 lbs., and one channel 15" @ 33 lbs. The ⅜ in. plate floor is carried on 12" I-beams spaced about 1' 6" centers. The steel plate floor is placed at a slope of 8 in. in 12 in., and it is stated that 95 per cent of the coal can be withdrawn from the bin. The bins discharge through vertical gates

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in the sides into motor-driven larries, which run to the coke ovens. The vertical gates are raised by rack and pinion and chain wheels.

Structural steel used in coal tipples for coking plants is especially liable to corrosion. Many of the shafts having head frames are "up-

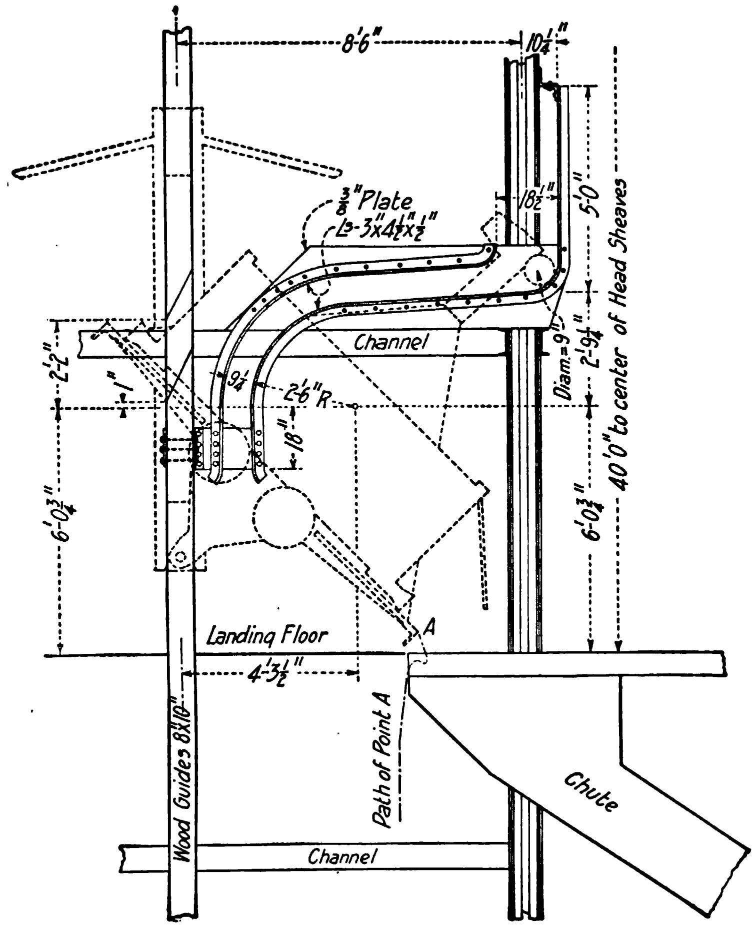

FIG. 144 d. DETAILS OF TIPPING GUIDES FOR SELF-DUMPING CAGE.

casts," and the air coming from these shafts is generally warmer than the outside air and carries considerable moisture and diluted gases. In order to make a structure anything like permanent the metal should have a minimum thickness of $\frac{5}{16}$ in. In the Phillips coal tipple the

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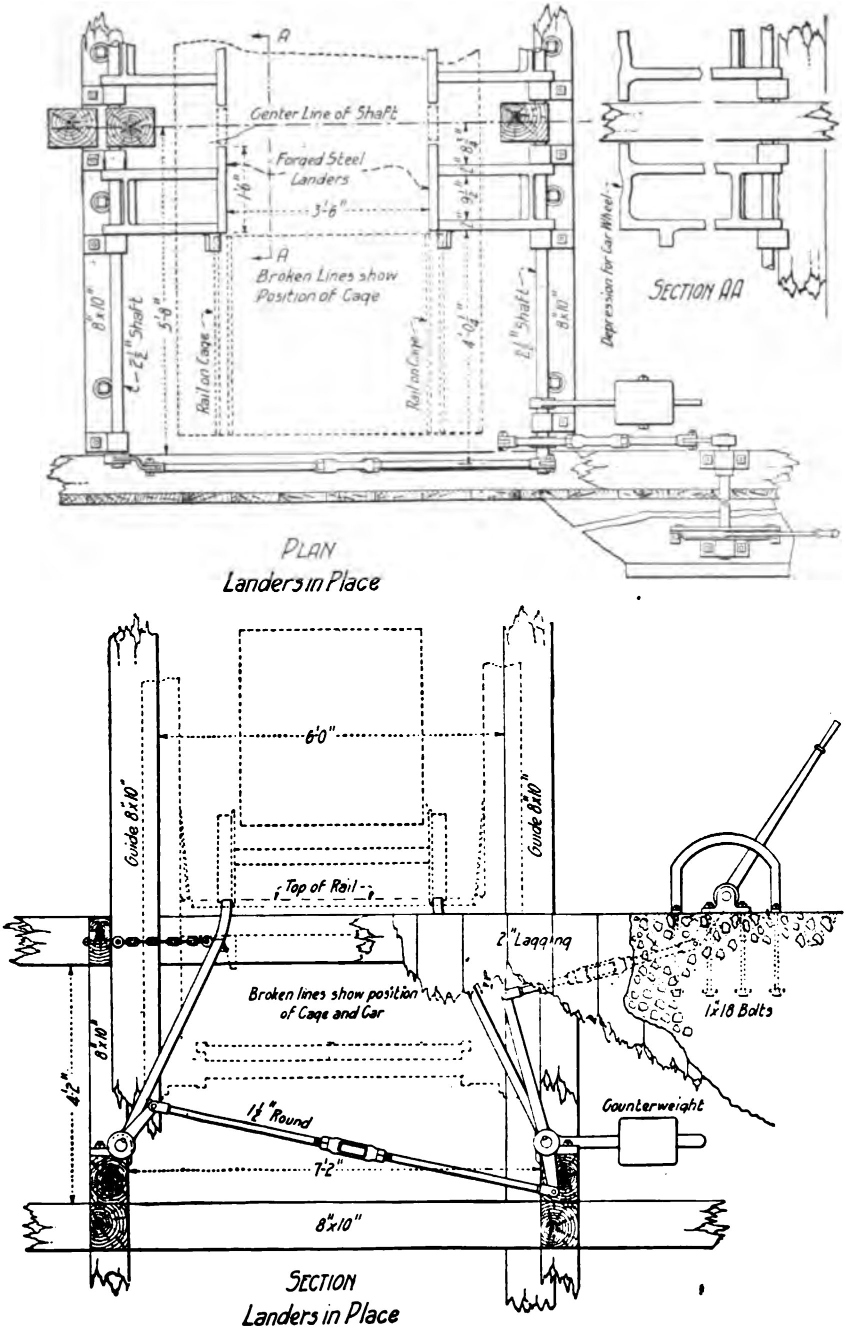

14 Fig. 144 e. Details of Landers for Self-dumping Cage. (193)

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main members have a minimum thickness of ⅜ in. in all members carrying stress to allow for corrosion. In this way it is assumed that the life of the steel coal tipple, which in this region has been approximately 15 years for steel structures designed with ¼ in. metal, will be very much increased. The steel used in this coal tipple is medium open hearth steel, manufactured under standard specifications. The steel was painted with one coat of graphite paint in the shop and with one coat of graphite paint after erection.

The mine buildings at this plant have walls of hard burned brick laid in cement-lime mortar, the roofs are carried on steel fink trusses and are covered with No. 1 ribbon Bangor slates fastened with copper nails and flashed with 16 oz. copper, and laid on beaded pine sheathing. The floor of the boiler house is made of 12 in. × 12 in. clay oven tile, laid on a 6 in. foundation of fine coke ash, the joints being grouted with cement mortar.

The hoisting rope is 1⅝ in. crucible cast steel; depth of shaft, 268 feet; height of dumping point above the surface, 80 feet; distance from center of drum to center of shaft, 84 ft. 5⅜ in. The mine cars are steel, with a capacity of fifty bushels of coal, and are hauled to the bottom of the shaft by one 10-ton and two 12-ton compressed air locomotives; the locomotives being charged from a pipe line carrying a pressure of 1,000 pounds per sq. in.

Self-dumping Cages.—The self-dumping cages used in the Phillips mine are shown in Fig. 144 a to 144 c. The mine cars have a gage of 3 ft. 6 in., and a capacity of 50 bushels of coal. The details of the tipping guides are shown in Fig. 144 d; the details of the landers are shown in Fig. 144 e. The details of the operation of the self-dumping cages may be followed by studying the detail plans. The self-dumping cage just described is the standard cage of the H. C. Frick Coke Company.

A general view of the Filbert steel tipple is shown in Fig. 16 and Fig. 17.

A Timber Coal Tipple.—The detail plans of a timber coal tipple designed for the Consolidated Coal Co., Collinsville, Ill., by the Link Belt Machinery Co., is shown in Fig. 144 f, and the tipple equipment is shown in Fig. 144 g. The head frame is 65 ft. 6 in. from the collar of the shaft to the center of the sheaves, which are 8 ft. in diameter. The main members of the head frame are made of 2 pieces 6″ × 12″, bolted together with ⅝ in. bolts as shown; and the bracing is made of

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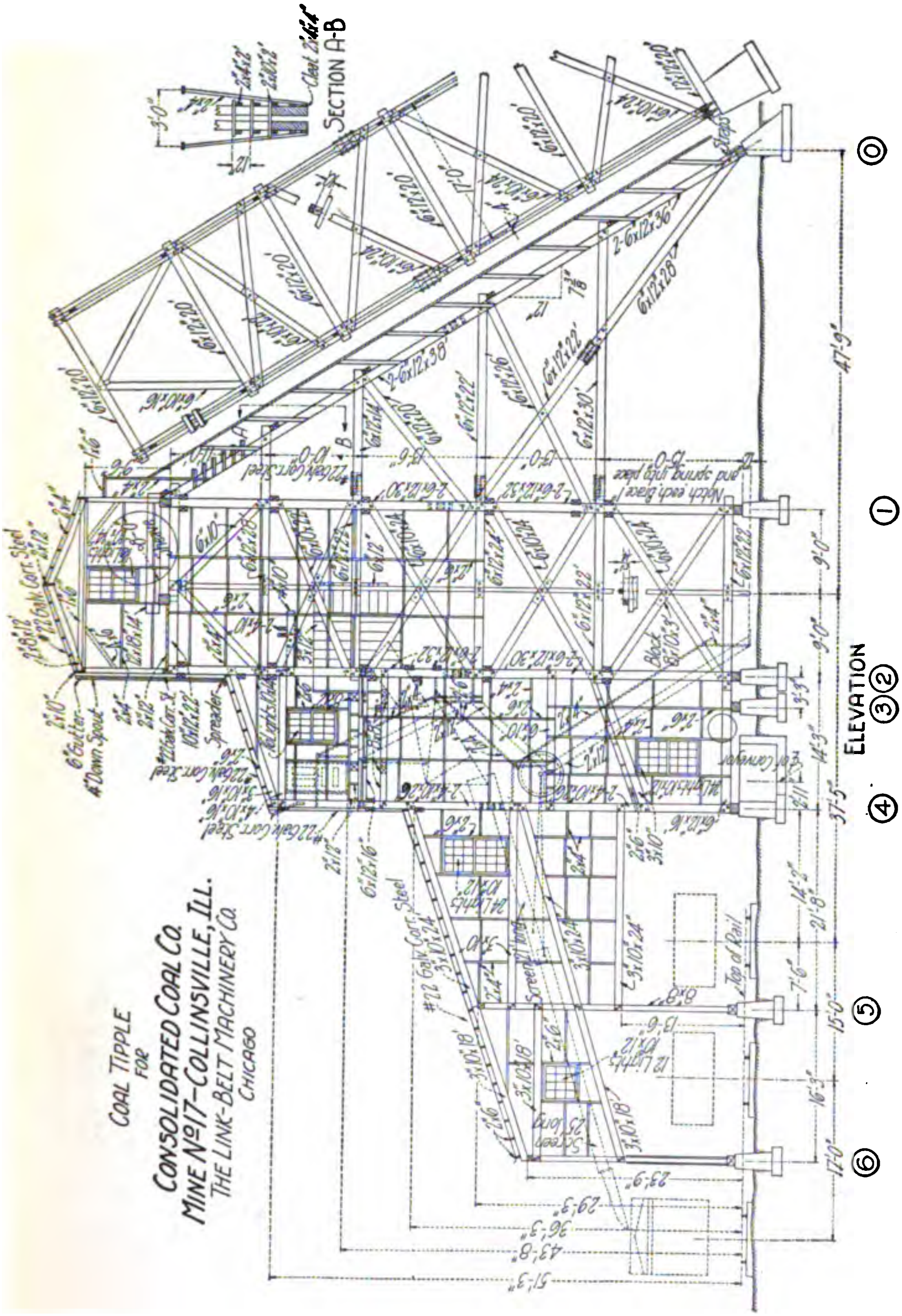

Fig. 144 f.

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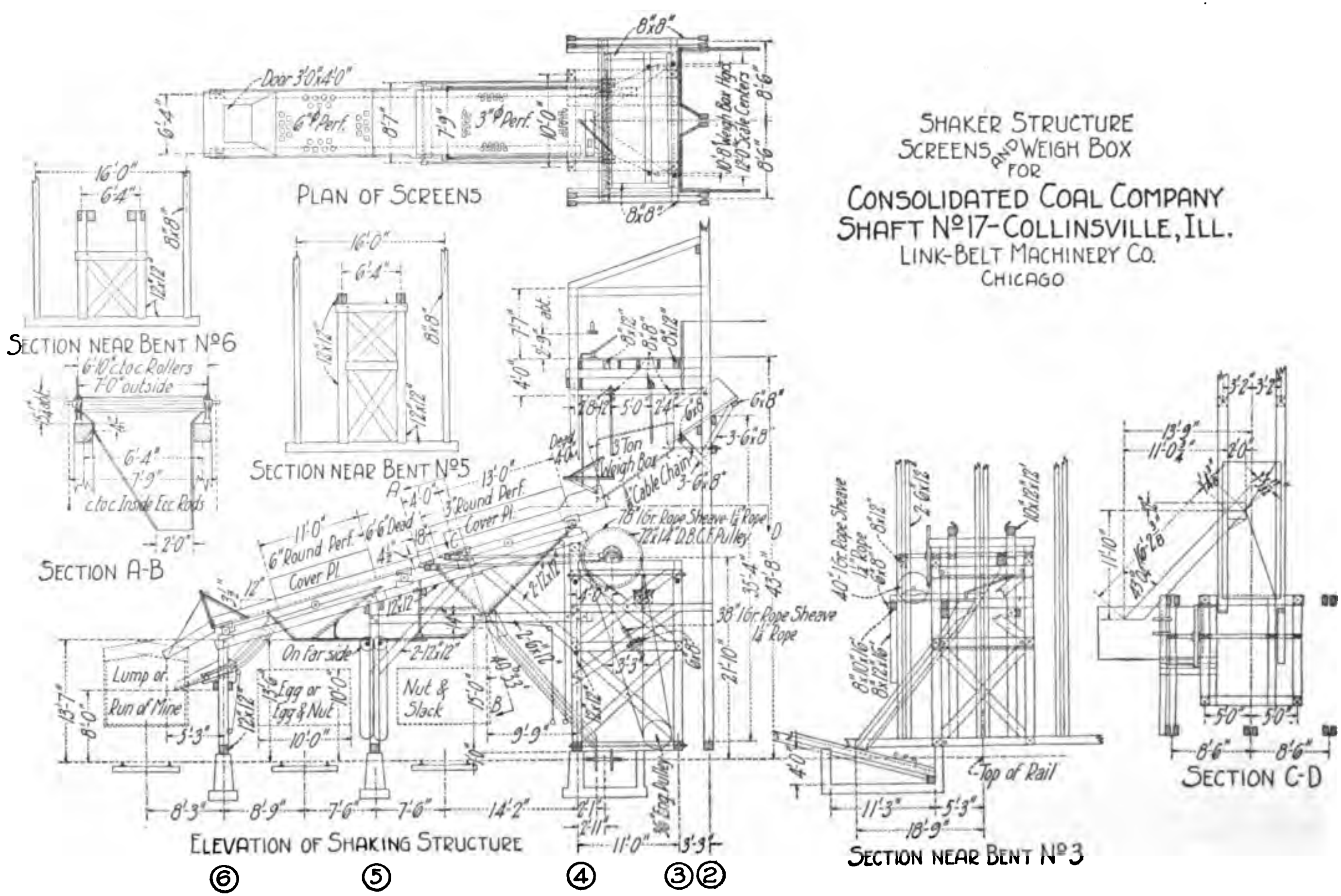

SHAKER STRUCTURE
SCREENS AND WEIGH BOX
FOR
CONSOLIDATED COAL COMPANY
SHAFT No 17-COLLINSVILLE, ILL.
LINK-BELT MACHINERY CO.
CHICAGO
PLAN OF SCREENS
SECTION NEAR BENT No 6
SECTION NEAR BENT No 5
SECTION A-B
ELEVATION OF SHAKING STRUCTURE
SECTION NEAR BENT No 3
SECTION C-D
Door 3'0"x4'0"
6"φ Perf.
3"φ Perf.
10'8" Weigh Box Hgrs.
12'0" Scale Centers
6'10" c. to c. Rollers
7'0" outside
c. to c. Inside Ecc. Rods
6" Round Perf.
3" Round Perf.
Cover Pl.
3 Ton Weigh Box
Cable Chain
78" 16 Gr. Rope Sheave 1¼" Rope
72"x14" D.B.C.F. Pulley
38" 16 Gr. Rope Sheave 1¼" Rope
40" 16 Gr. Rope Sheave 1¼" Rope
96" Eng. Pulley
On far side
Lump or Run of Mine
Egg or Egg & Nut
Nut & Slack
Top of Rail

Fig. 144 g.

pieces 6″ × 12″. The columns for the shaker building are made of 8″ × 8″ timbers. The girts are 2″ × 4″, while the purlins are 2″ × 6″. The structure is covered on the roof and sides with No 22 galvanized corrugated steel nailed to the purlins and girts with 8 d. barbed roofing nails.

The shaking screens are carried on an independent structure, as shown in Fig. 144 g. The main members of the shaker structure are made of 12″ × 12″ timbers, fully braced as shown. The coal is raised in self-dumping cages and is dumped into a 3-ton weigh box, from which it runs over the shaking screens. The screens are carried on rollers and are driven by eccentrics. The shaking screens are set at a slope of 3½ in. in 12 in.

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PART II.

THE DESIGN OF MINE BUILDINGS.

Introduction.—This part of the book includes a brief discussion of the design of roof trusses, steel frame buildings, bins and retaining walls, and the design of miscellaneous mine structures including coal washers and coal breakers. The treatment of steel frame buildings and bins is necessarily brief, the reader being referred to the author's "The Design of Steel Mill Buildings" and "The Design of Walls, Bins and Grain Elevators" for a more complete treatment. Data on design of mine buildings will be found in Chapter XIV, and also in "The Specifications for Steel Mine Structures" in Appendix I.

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CHAPTER VIII.

Stresses in Roof Trusses and Frame Structures.

STRESSES IN ROOF TRUSSES. Loads.—The stresses in roof trusses are due to (1) the dead load, (2) the snow load, (3) the wind load, and (4) concentrated and moving loads. Data for dead loads, snow loads, wind loads, crane loads and other loads to be carried on roof trusses are given in "The Specifications for Steel Mine Structures" in Appendix I. The loads on roof trusses are commonly given as a certain number of lbs. per sq. ft. of horizontal projection of the roof. The loads are assumed to be transferred to the truss by means of purlins acting as simple beams, the joint loads being equal to the purlin reactions.

Methods of Calculation.—The determination of the reactions of simple framed structures usually requires the use of the three fundamental equations of equilibrium

$$\Sigma \text{ horizontal components of forces} = 0 \qquad (a)$$

$$\Sigma \text{ vertical components of forces} = 0 \qquad (b)$$

$$\Sigma \text{ moments of forces about any point} = 0 \qquad (c)$$

Having completely determined the external forces, the internal stresses may be obtained by either equations (*a*) and (*b*) (resolution), or equation (*c*) (moments). These equations may be solved by graphics or by algebra. There are, therefore, four methods of calculating stresses:

$$\text{Resolution of Forces} \begin{cases} \text{Graphic Method} \\ \text{Algebraic Method} \end{cases}$$

$$\text{Moments of Forces} \begin{cases} \text{Graphic Method} \\ \text{Algebraic Method} \end{cases}$$

The stresses in any simple framed structure can be calculated by using any one of the four methods. The method of calculating the

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stresses in roof trusses by means of graphic resolution will be explained in detail. For the calculation of the stresses in roof trusses and other framed structures by algebraic resolution and by algebraic and graphic moments the reader is referred to the author's "The Design of Steel Mill Buildings." The stresses in a portal and a trestle bent are calculated both by graphic resolution and algebraic moments in Fig. 153 and Fig. 154, respectively; while the stresses in a roof truss are calculated in Fig. 156b, both by graphic and algebraic resolution.

Graphic Resolution.—In Fig. 145 the reactions R_1 and R_2 are found by means of the force and equilibrium polygons as shown in (b) and (a). The principle of the force polygon is then applied to each joint of the structure in turn. Beginning at the joint L_0, the forces are shown in (c), and the force triangle in (d). The reaction R_1 is known and acts up, the upper chord stress 1–x acts downward to the left, and the lower chord stress 1–y acts to the right, closing the polygon. Stress 1–x is compression and stress 1–y is tension, as can be seen by applying the arrows to the members in (c). The force polygon at joint U_1 is then constructed as in (f). Stress 1–x acting toward joint U_1 and load P_1 acting downward are known, and stresses 1–2 and 2–x are found by completing the polygon. Stresses 2–x and 1–2 are compression. The force polygons at joints L_1 and U_2 are constructed, in the order given, in the same manner. The known forces at any joint are indicated in direction in the force polygon by double arrows, and the unknown forces are indicated in direction by single arrows.

The stresses in the members of the right segment of the truss are the same as in the left, and the force polygons are, therefore, not constructed for the right segment. The force polygons for all the joints of the truss are grouped into the stress diagram shown in (k). Compression in the stress diagram and truss is indicated by arrows acting toward the ends of the stress lines and toward the joints, respectively, and tension is indicated by arrows acting away from the ends of the stress lines and away from the joints, respectively. The first time a stress is used a single arrow, and the second time the stress is used a

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double arrow is used to indicate direction. The stress diagram in (*k*) Fig. 145 is called a Maxwell diagram or a reciprocal polygon diagram, *i. e.*, areas in the truss diagram become points in the stress diagram.

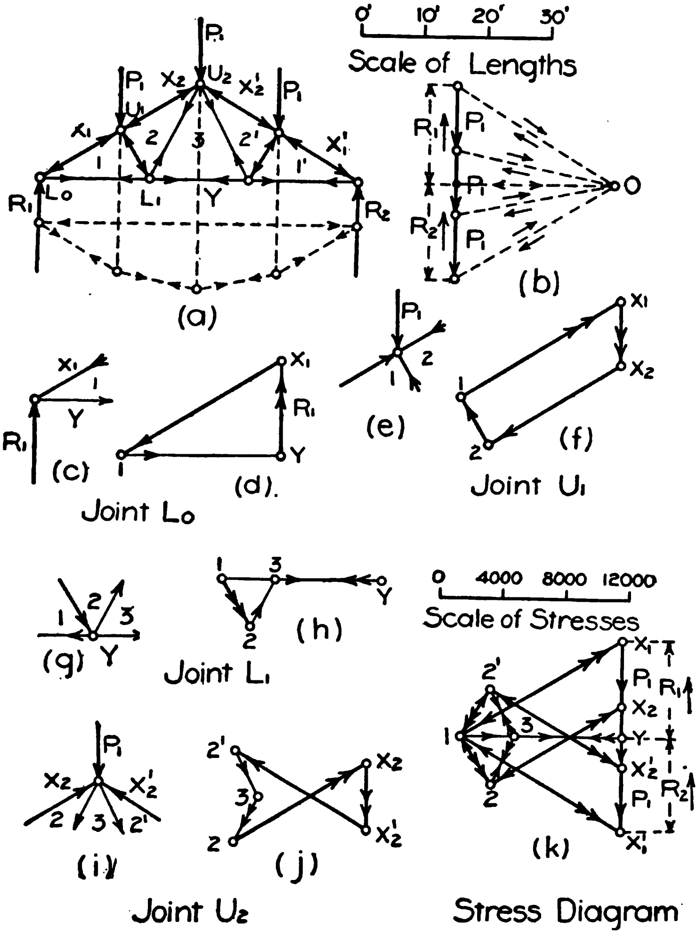

FIG. 145

The notation used is known as Bow's notation. The method of graphic resolution is the method most commonly used for calculating stresses in roof trusses and in simple framed structures with inclined chords.

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Loads.—Data for loads are given in "Specifications for Steel Mine Structures," Appendix I. A formula for weights of steel trusses is given in § 13; snow loads are given in § 19; wind loads are given in § 20; and weights of roof covering are given in § 14 to § 18. The weights of cranes are given in Table XXVIII and Table XXIX, Chapter XIV.

Dead Load Stresses.—The dead load is made up of the weight of the truss and the roof covering, and is usually considered as applied at

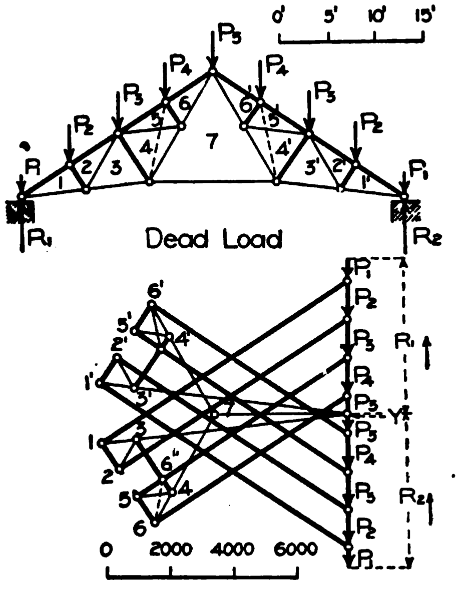

FIG. 146. DEAD LOAD STRESS DIAGRAM.

the panel points of the upper chords in computing stresses in roof trusses. If the purlins do not come at the panel points, the upper chord will have to be designed for direct stress and stress due to flexure.

The stresses in a Fink truss due to dead loads are calculated by graphic resolution in Fig. 146.

The loads are laid off, the reactions found, and the stresses calculated beginning at joint L_0, as explained in Fig. 145. The stress diagram for the right half of the truss need not be drawn where the truss and

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loads are symmetrical as in Fig. 146; however, it gives a check on the accuracy of the work and is well worth the extra time required. The loads P_1 on the abutments have no effect on the stresses in the truss, and may be omitted in this solution.

In calculating the stresses at joint P_8, the stresses in the members 3–4, 4–5 and x–5 are unknown, and the solution appears to be indeterminate. The solution is easily made by cutting out members 4–5 and 5–6, and replacing them with the dotted member shown. The stresses in the members in the modified truss are now obtained up to and including stresses 6–x and 6–7. Since the stresses 6–x and 6–7 are independent of the form of the framework to the left, as can easily be seen by cutting a section through the members 6–x, 6–7 and 7–y, the solution can be carried back and the apparent ambiguity removed. The ambiguity can also be removed by calculating the stress in 7–y by algebraic moments and substituting it in the stress diagram. It will be noted that all top chord members are in compression and all bottom chord members are in tension.

Dead and Ceiling Load Stresses.—The stresses in a triangular Pratt truss due to dead and ceiling loads, are calculated by graphic resolution in Fig. 147.

For simplicity the stresses are shown for one side only. The reaction R_1 is equal to one half of the entire load on the truss. The solution will appear more clear when it is noted that the stress diagram shown consists of two diagrams, one due to loads on the upper chord and the other due to loads on the lower chord, combined in one, the loads in each case coming between the stresses in the members on each side of the load. The top chord loads are laid off in order downward, while the bottom chord loads are laid off in order upward.

Snow Load Stresses.—Large snow storms nearly always occur in still weather, and the maximum snow load will therefore be a uniformly distributed load. A heavy wind may follow a sleet storm and a snow load equal to the minimum given in § 19, " Specifications for Steel Mine Structures," Appendix I, should be considered as acting at the same

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time as the wind load. The stresses due to snow load are found in the same manner as the dead load stresses.

Wind Load Stresses.—The stresses in trusses due to wind load will depend upon the direction and intensity of the wind, and the condition of the end supports. The wind is commonly considered as act-

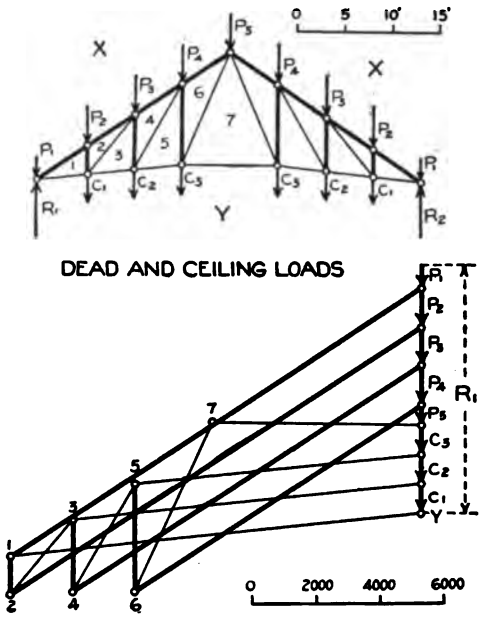

FIG. 147. STRESS DIAGRAM FOR ROOF AND CEILING LOADS.

ing horizontally, and the normal component, as determined by one of the formulas in § 20, "Specifications for Steel Mine Structures," Appendix I, is taken.

The ends of the truss may (1) be rigidly fixed to the abutment walls, (2) be equally free to move, or (3) may have one end fixed and the other end on rollers. When both ends of the truss are rigidly fixed to the abutment walls (1) the reactions are parallel to each other and to the resultant of the external loads; where both ends of the truss are equally free to move (2) the horizontal components of the reactions are equal; and where one end is fixed and the other end is on friction-

less rollers (3) the reaction at the roller end will always be vertical. Either case (1) or case (3) is commonly assumed in calculating wind load stresses in trusses. Case (2) is the condition in a portal, Fig. 153, or framed bent, Fig. 152. The vertical components of the reactions are independent of the condition of the ends.

Wind Load Stresses: No Rollers.—The stresses due to a normal wind load, in a Fink truss with both ends fixed to rigid walls, are calculated by graphic resolution in Fig. 148. The reactions are parallel and

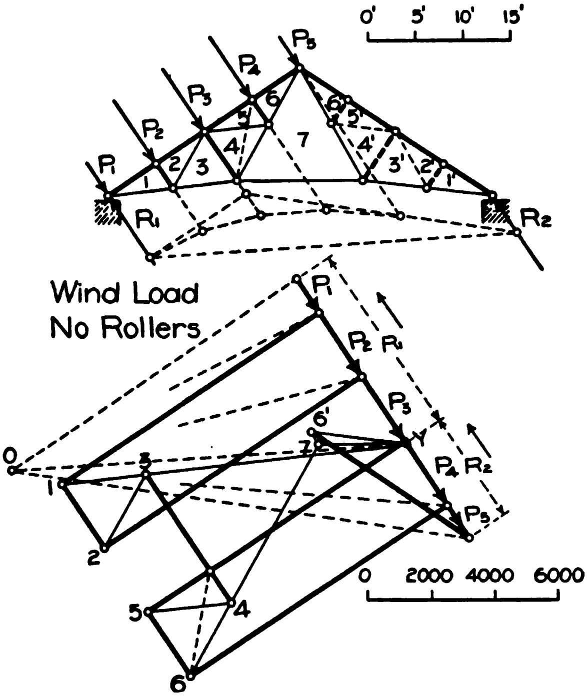

FIG. 148. STRESS DIAGRAM FOR WIND LOADS.

their sum equals the sum of the external loads; they are found by means of force and equilibrium polygons. To calculate the reactions, lay off the loads P_1, P_2, P_3, P_4, P_5, as shown, and select the pole O at any convenient point. Then at a point on line of action of P_1 in the truss diagram, draw strings parallel to the rays drawn through the

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ends of P_1 in the force polygon. The string drawn parallel to the ray common to forces P_1 and P_2 in the force polygon will cut the force P_2 in the truss diagram. Through this point draw a string parallel to the ray common to forces P_2 and P_3 in the force polygon, and so on until the strings drawn parallel to the outside rays meet on the resultant of all the loads. The closing line of the force polygon connects the two points on the reactions. Through point O in the force polygon draw line O–Y parallel to the closing line in the equilibrium polygon, R_1 and R_2 are the reactions, as shown.

The stress diagram is constructed in the same manner as that for dead loads. Heavy lines in truss and stress diagram indicate compression, and light lines indicate tension.

The ambiguity at joint P_3 is removed by means of the dotted member, as in the case of the dead load stress diagram. It will be seen that there are no stresses in the dotted web members in the right segment of the truss. It is necessary to carry the solution entirely through the truss, beginning at the left reaction and checking up at the right reaction. It will be seen that the load P_1 has no effect on the stresses in the truss in this case, the left reaction being simply reduced by P_1.

Wind Load Stresses: Rollers.—Trusses longer than 70 ft. are usually fixed at one end, and are supported on rollers at the other end. The reaction at the roller end is then vertical—the horizontal component of the external wind force being all taken by the fixed end. The wind may come on either side of the truss, giving rise to two conditions: (1) rollers leeward and (2) rollers windward, each requiring a separate solution.

Rollers Leeward.—The wind load stresses in a triangular Pratt truss with rollers under the leeward side are calculated by graphic resolution in Fig. 149.

The reactions in Fig. 149 were first determined by means of force and equilibrium polygons, on the assumption that they were parallel to each other and to the resultant of the external loads. Then since the reaction at the roller end is vertical and the horizontal component at the

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fixed end is equal to the horizontal component of the external wind forces, the true reactions were obtained by closing the force polygon.

In order that the truss be in equilibrium under the action of the three external forces, R_1, R_2 and the *resultant of the wind loads,* the three external forces must meet in a point if produced. This furnishes a method for determining the reactions, where the direction and line of action of one and a point in the line of action of the other are known,

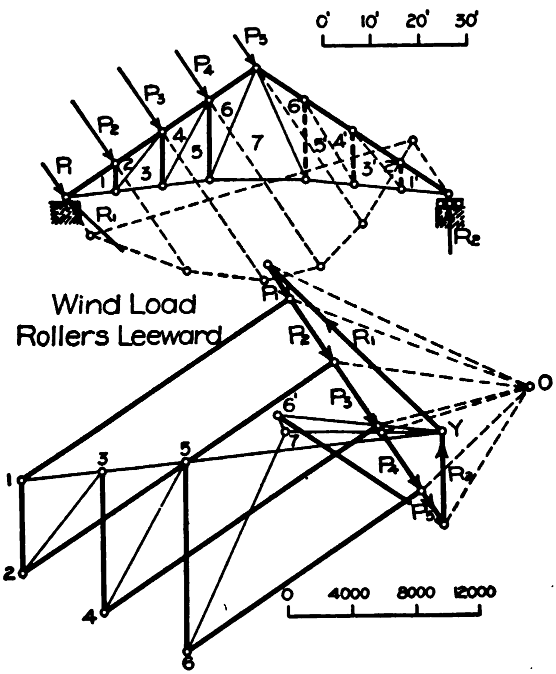

FIG. 149. STRESS DIAGRAM FOR WIND LOADS.

providing the point of intersection of the three forces comes within the limits of the drawing board.

The stress diagram is constructed in the same way as the stress diagram for dead loads. It will be seen that the load P_1 has no effect on the stresses in the truss in this case. Heavy lines in truss and stress diagram indicate compression, and light lines indicate tension.

Rollers Windward.—The wind load stresses in the same triangular

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Pratt truss as shown in Fig. 149, with rollers under the windward side of the truss are calculated by graphic resolution in Fig. 150.

The true reactions were determined directly by means of force and equilibrium polygons. The direction of the reaction R_1 is known to be vertical, but the direction of the reaction R_2 is unknown, the only known point in its line of action being the right abutment. The equilibrium polygon is drawn to pass through the right abutment and the direction of the right reaction is determined by connecting the point of

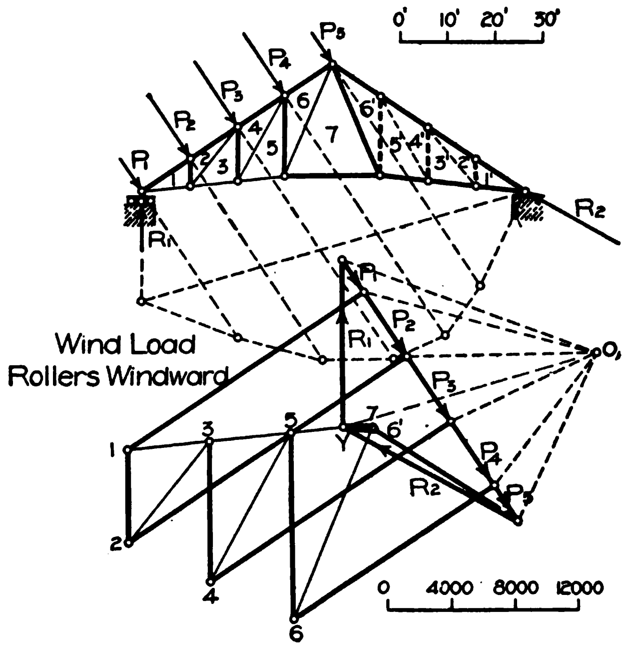

FIG. 150. STRESS DIAGRAM FOR WIND LOADS.

intersection of the vertical reaction R_1 and the line drawn through O parallel to the closing line of the equilibrium polygon, with the lower end of the load line.

Since the vertical components of the reactions are independent of the conditions of the ends of the truss, the vertical components of the reactions in Fig 149 and Fig. 150 are the same. It will be seen that

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the load P_1 produces stress in the members of the truss with rollers windward. If the line of action of R_2 drops below the joint P_5, the lower chord of the truss will be in compression, as will be seen by taking moments about P_5.

Concentrated Load Stresses.*—The stresses in a Fink truss due to unequal crane loads are calculated by graphic resolution in Fig. 151.

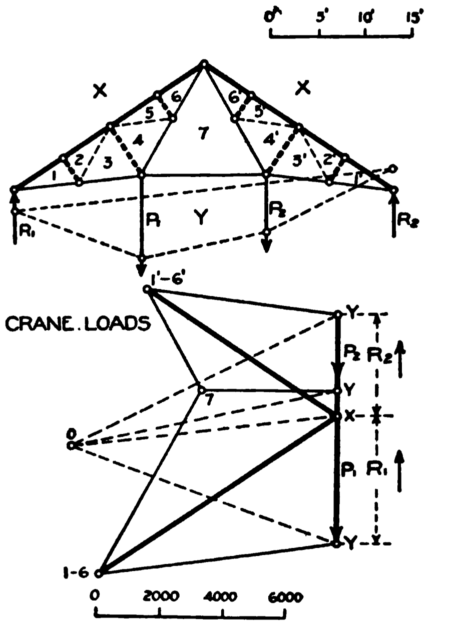

FIG. 151. STRESS DIAGRAM FOR CONCENTRATED LOADS.

The reactions were found by means of force and equilibrium polygons. The truss is reduced to three triangles for the loading shown. The solution of this problem is similar to that for ceiling loads in Fig. 147. The moving crane trolley will produce maximum moment when it is at the center of the truss, and this case should also be investigated in solving the problem.

* Weights of cranes are given in Table XXVIII and Table XXIX, Chapter XIV.

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STRESSES IN A TRANSVERSE BENT.—A transverse bent in a steel mill building consists of a roof truss supported at the ends on columns and braced against longitudinal movement by means of knee braces, Fig. 158. The ends of the columns may be fixed at the base or may be free to turn (pin-connected). The stresses in a transverse bent are statically indeterminate and cannot be calculated without taking in account the deformations of the members themselves. The following approximate method, proposed by the author, gives results that are approximately correct, are on the safe side, and is the method used in practice.

Dead and Snow Load Stresses.—The stresses due to dead and snow loads in the truss of a transverse bent are calculated the same as though the truss were supported on solid walls.

Wind Load Stresses.—The external wind loads may be taken (1) as horizontal or (2) as normal to the surface. The columns will be assumed to be pin-connected at the tops and to be either pin-connected or fixed at the base. It will be assumed that the horizontal reactions at the foot of the columns are equal to each other, and equal to ½ of the horizontal component of the external wind load. It is also assumed that the truss does not change its length, and that the deflection of the columns at the top of the columns and at the foot of the knee brace are equal.

It is shown in "The Design of Steel Mill Buildings" that when the columns are fixed at the base the point of contra-flexure comes at a distance of from ½ to ⅝ of the distance from the foot of the column to the foot of the knee brace. It is usually assumed that the point of contra-flexure is located at a point in the column one half the distance from the foot of the column to the foot of the knee brace. (For the calculation of the point of contra-flexure in the columns of a transverse bent, see Fig. 72, Chapter IV.)

Graphic Calculation of Stresses.—The graphic calculation of the stresses in transverse bents will be illustrated by a problem, Fig. 152.

Problem.—Calculate the stresses due to wind loads in a transverse

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bent, span 40′ 0″; pitch of roof $\frac{1}{4}$; height of posts 20′ 0″; post pin connected at the base; wind load 20 lbs. per sq. ft. of vertical projection and assumed to act horizontally on the roof and sides, Fig. 152.

Solution.—$H = \frac{1}{2}P = 4{,}500$ lbs. $= H'$. To calculate V take moments about the foot of the right-hand post, and $V \times 40' - 3{,}000 \times 13\frac{1}{3}' - 1{,}750 \times 20' - 1{,}500 \times 25' - 750 \times 30' = 0$. Then $V = + 3{,}375$ lbs. $= - V'$.

To construct the stress diagram lay off the load line $P_1 + P_2 + P_3 + P_4 + P_5$ and $1'\text{–}Y = V = 3{,}375$ lbs. Beginning at the foot of the windward post, V acts downward, $H = X\text{–}1$ acts to the left, P_5 acts to the right. The polygon is closed by drawing lines parallel to $1\text{–}X$ and $1\text{–}Y$, the final stress polygon being $Y\text{–}1\text{–}X\text{–}X\text{–}1'$. Then pass to the load P_4 in the transverse bent, and in the stress diagram P_4 acts to the right, $1\text{–}X$ acts upwards to the left, $1\text{–}2$ acts to the right, and $2\text{–}X$ acts downward to the left, closing the polygon. The remainder of the stress diagram is drawn in a similar manner, passing to the foot of the knee brace, then to the top of the post, etc., finally checking up at the foot of the leeward post. The maximum shear is in the leeward post; below the knee brace the shear is $H = 4{,}500$ lbs., above the knee brace the shear is the horizontal component of the stress in $10\text{–}X = 10'\text{–}X = 9{,}000$ lbs. The maximum bending moment in the post is at the foot of the leeward knee brace, and is $M = 4{,}500 \times 13\frac{1}{3}' = 60{,}000$ ft.-lbs.

It will be seen that the stresses in the members do not follow the usual rules for trusses loaded with vertical loads; the top chord is partly in compression and partly in tension, while the bottom chord is in compression. The bent should be designed for the wind load stresses combined with the dead load and the minimum snow load stresses, for the wind load and dead load stresses, or for the dead load and snow load, whichever combination produces maximum stresses or reversals of stresses.

The stresses in the posts are calculated by dropping points 1, 2, 10, 11 to the 1′, 2′, 10′ and 11′, respectively, on the load line, or on load line produced. The stresses in the windward post are $1'\text{–}Y$ and $2'\text{–}3$,

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while the stresses in the leeward post are *11′–Y* and *9–10′*. The maximum shear in the leeward post is above the knee brace and is *10′–X* = 9,000 lbs.

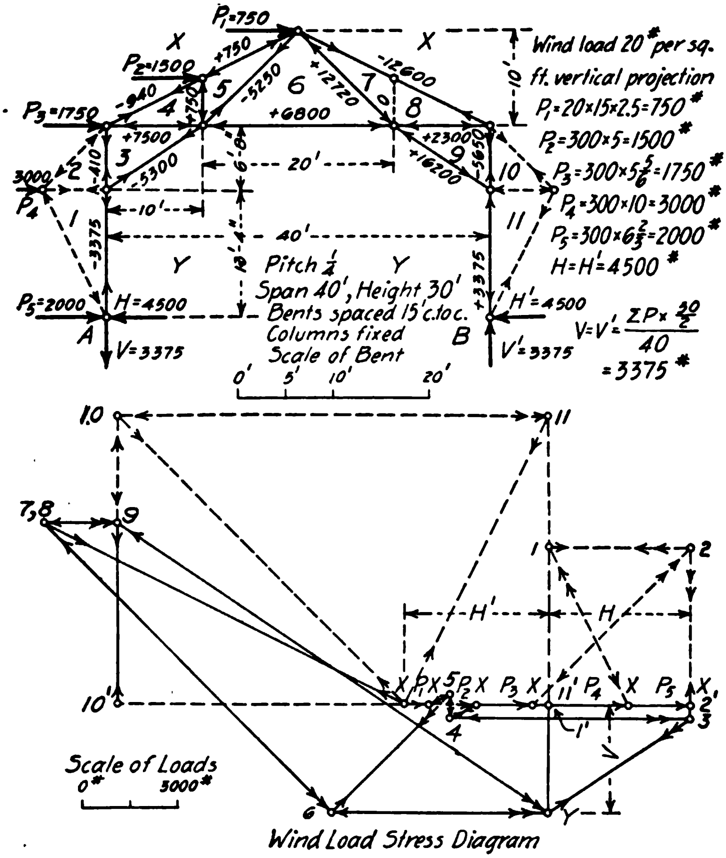

FIG. 152. WIND LOAD STRESS DIAGRAM FOR TRANSVERSE BENT.

Where the columns are fixed at the base the posts are assumed as being pin-connected at the point of contra-flexure and the stresses are calculated for the loads upon the bent above the plane through the point of contra-flexure in the manner above described.

For a more complete discussion of the stresses in transverse bents and in portals, see the author's "The Design of Steel Mill Buildings."

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STRESSES IN A PORTAL.—The stresses in a portal will be calculated by algebraic moments and graphic resolution. The portal in Fig. 153 has a height of 30′ 0″, a center to center width of 15′ 0″, is pin-connected (free to turn) at the base, and carries a load of $R = 2{,}000$

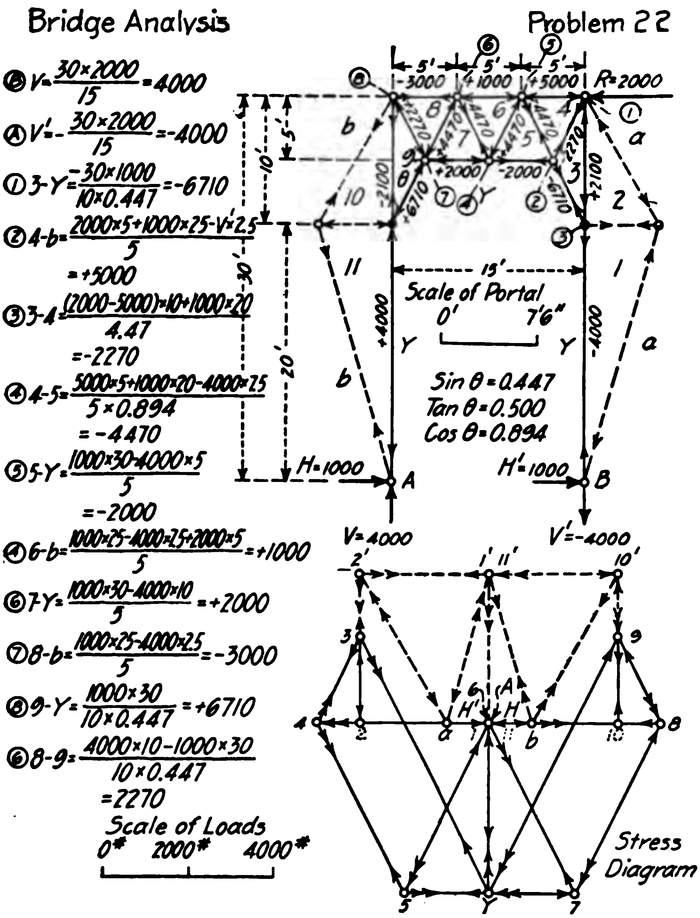

FIG. 153. STRESSES IN PORTAL BRACING.

lbs. (Note. The portal is not pin-connected at joint (3), and the corresponding joint on the other side, as might be inferred from Fig. 153.)

Solution.—Now $H = H' = 1{,}000$ lbs., $V = -V'$, and by taking moments about B, $V = 30 \times 2{,}000 \div 15 = 4{,}000$ lbs. $= -V'$.

Algebraic Moments.—In passing sections, care should be used to avoid cutting the columns for the reason that these members are subject to bending stresses in addition to the direct stresses. To calculate the stress in member $3-Y$, take the center of moments at joint (1) and pass a section cutting members $4-b$, $3-4$ and $3-Y$, and cutting the portal away to the left of the section. Then assume stress $3-Y$ as an external force acting from the outside toward the cut section, and $3-Y \times 10 \times 0.477\ (\sin\theta) + H \times 30 = 0$. The stress in $3-Y = -6{,}170$ lbs. The remaining stresses are calculated as shown.

Graphic Resolution.—Lay off $a-A = A-b = H = 1{,}000$ lbs., and $A-Y = V' = 4{,}000$ lbs. Then beginning at point B in the portal the force polygon for equilibrium is $a-A-Y-1'-a$, in which $1'-a$ is the stress in the auxiliary member $1-a$, and $Y-1'$ is the stress in the post $1-Y$ when the auxiliary member is acting. The true stress in $1-Y$ is equal to the algebraic sum of the vertical components of the stress $1'-a$ and $1'-Y$, and equals $V' = -4{,}000$ lbs. Next complete the force triangle at the intersection of the auxiliary members. Stress $1'-a$ is known, and the force triangle is $a-1'-2'-a$, the forces acting as shown. The stress diagram is carried through in the order shown, checking up at the point A. The correct stresses are shown by the full lines in the stress diagram. The true stress in $3-2$ will produce equilibrium for vertical stresses at joint (1) as shown. The maximum shear in the posts is $H = 1{,}000$ lbs. below the knee-brace and 2,000 lbs. above the knee-brace. The maximum bending moment in the posts will occur at the foot of the member $3-Y$, joint (3), and is $M = 1{,}000 \times 20 \times 12 = 240{,}000$ in.-lbs.

STRESSES IN A TRESTLE BENT.—Given a trestle bent, Fig. 154, height 45′ 0″, width at the base 30′ 0″, width at the top 9′ 0″, wind loads P_0, P_1, P_2, P_3, P_4, as shown. The diagonals are tension members, the dotted members not being stressed for the wind blowing as shown. The stresses will be calculated by graphic resolution and by algebraic moments.

1. **Graphic Resolution.**—The load P_0 is assumed as transferred to the bent by means of auxiliary members as shown. The loads P_0, P_1,

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P_2, P_3, P_4 are laid off as a load line as shown. Beginning with P_0, the load is held in equilibrium by means of the auxiliary members, and the load P_0 is equilibrated in the stress diagram by drawing *1–2* and *Y–2* parallel to dotted members *1–2* and *Y–2*, the stresses in the stress diagram acting around to the right, stress *1–2* is tension and *Y–2* is compression. The load P_1 on the bent is held in equilibrium by the auxil-

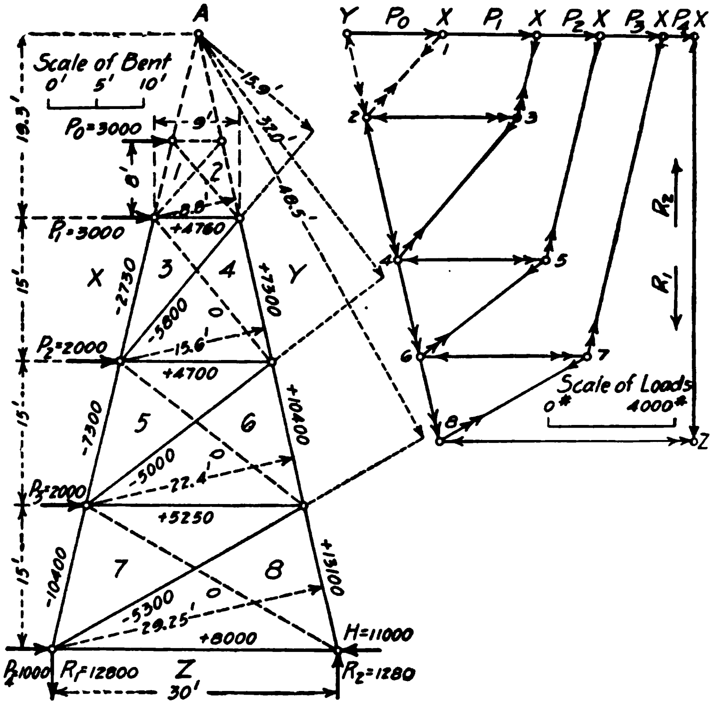

FIG. 154. WIND STRESSES IN A TRESTLE BENT.

iary stress *1–2* and stresses in members *2–3* and *x–3* not yet calculated. The stress polygon is closed by drawing *2–3* and *x–3* parallel to members *2–3* and *x–3*. The stresses act around the stress polygon *x–3–2–1* to the right, *x–3* being tension and *2–3* being compression. The stresses in auxiliary member *Y–2* and member *2–3* are held in equilibrium by stresses in members *3–4* and *Y–4*. In the stress diagram *2–3* is compression acting to the right, *Y–2* is compression acting downward, and

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the stress polygon is closed by drawing *Y–4*, compression, and *3–4*, tension. It will be seen that for equilibrium the stresses all act in the same direction around the stress polygon. The first time a stress is used in the stress diagram, one arrow is used, and the second time the stress is used two arrows are used to indicate direction and kind of stress. The remainder of the solution is easily followed.

2. **Algebraic Moments.**—To calculate the stresses in the diagonal members take centers of moments about the point *A*, the point of intersection of the inclined posts Then to calculate the stress in *3–4*, pass a section cutting members *3–X*, *3–4* and *4–Y*; assume that the stress in *3–4* is an external force acting from the outside toward the cut section, and $3\text{–}4 \times 15.9' + 3{,}000 \times 19.3' + 3{,}000 \times 11.3' = 0$. The stress $3\text{–}4 = -5{,}800$ lbs Stresses in *4–5*, *5–6*, *6–7*, *7–8* and *8–Z* are calculated in a similar manner. To obtain reaction R_1 take moments about R_2 and $R_1 \times 30' - 2{,}000 \times 15' - 2{,}000 \times 30' - 3{,}000 \times 45' - 3{,}000 \times 53' = 0$ Then $R_1 = 12{,}800$ lbs. $= -R_2$.

To calculate the stress in *4–Y*, take center of moments at joint P_2 and pass a section cutting members *5–x*, *4–5* and *4–Y*, and assume the stress in *4–Y* as an external force acting from the outside toward the cut section. Then $4\text{–}Y \times 15.6' - 3{,}000 \times 15' - 3{,}000 \times 23' = 0$. Then $4\text{–}Y = +7{,}300$ lbs.

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CHAPTER IX.

THE DESIGN OF ROOF TRUSSES AND STEEL FRAME BUILDINGS.

Truss Defined.—A truss is a framed structure in which the members are so arranged and fastened at their ends that external loads will cause only direct stresses in the members. In its simplest form a truss is a triangle or a combination of triangles. In this chapter it will be assumed (1) that the structure is not constrained by the reactions, (2) that the axes of the members meet in a common point at the joints, and (3) that the joints have frictionless hinges.

Types of Roof Trusses.—Several types of roof trusses are shown in Fig 155. These trusses have been subdivided so that the purlins will come at the panel points, and will not have a spacing greater than 4 ft. 9 in., the greatest spacing allowed for corrugated steel roofing when laid without sheathing (see Fig. 3, "Specifications for Steel Mine Structures"). The Fink trusses shown in (*a*) to (*g*) are commonly used in steel frame buildings and are very economical. The other types of trusses need no explanation.

Saw Tooth Roofs.—The common type of saw tooth roof is shown in (*a*), Fig. 156a. The glazed leg faces the north and permits only indirect light to enter the building, thus doing away with the glare and varying intensity of light in buildings where direct sunlight enters. In cold climates the snow drifts the gutters nearly full and causes loss of light and also leakage from the overflowing gutters. The modified saw tooth roof shown in (*b*) was designed by the author to obviate the defects in the common type of saw tooth roof. The modified saw tooth roof permits the use of a greater span and a more economical pitch than the common form shown in (*a*).

Pitch of Roof.—The pitch of a truss is expressed in terms of the center height divided by the span. For example, a truss with a 60 ft. span and a center height of 20 ft., has a pitch of $\frac{1}{3}$. For a corrugated steel roof the pitch should not be less than $\frac{1}{4}$, a pitch of $\frac{1}{3}$ being preferable. Shingles should have a pitch of $\frac{1}{3}$ to $\frac{1}{2}$. Tar and gravel roofs should preferably have a slope only sufficient to drain off the water.

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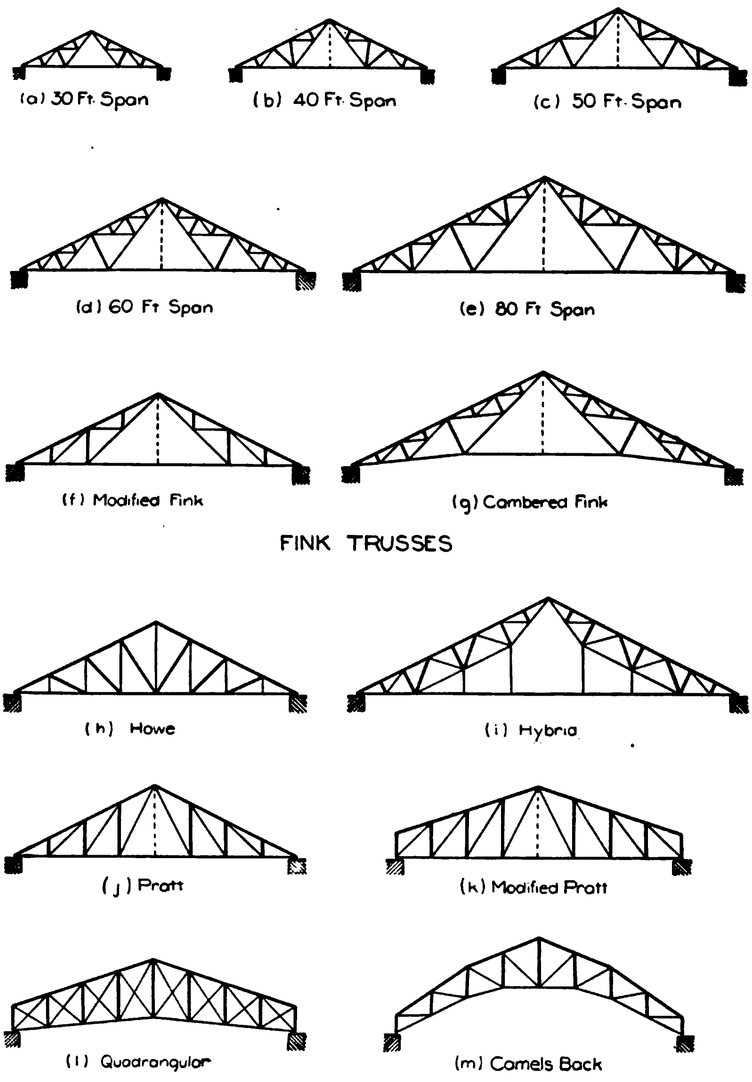

FIG. 155. TYPES OF ROOF TRUSSES.

Spacing of Trusses.—The spacing of trusses depends upon the span, the type of truss and the load to be carried. It is usually economical to use simple rolled sections for purlins, which makes a spacing of approximately 20 ft. a maximum. The spacing of trusses in mill

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buildings varies from 16 to 25 ft., the former spacing being a very satisfactory one up to spans of 60 to 80 ft.

Roof trusses for mill buildings are commonly made of angles riveted to connection plates at the joints. The truss for which the shop plans are shown in Fig. 227, Chapter XIV, has the main members

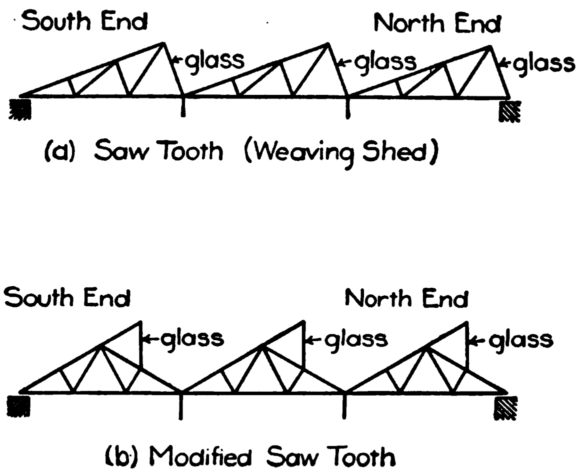

FIG. 156 a. SAW TOOTH TRUSSES.

made of two angles placed back to back, and the secondary members made of single angles. The members of a roof truss may be made of two channels or of two angles with a plate riveted between the angles. Where the purlins are not placed at the panel points the top chord must be designed for both direct stress and flexural stress.

DESIGN OF A STEEL ROOF TRUSS.—Given a Fink roof truss: span 40′ 0″, pitch 30 degrees, trusses spaced 12′ 0″, load 40 lbs. per sq. ft. of horizontal projection. The stresses have been calculated in Fig. 156b by algebraic resolution, and also by graphic resolution. The load P_1 is equal to $12 \times 10 \times 40$ lbs. $= 4{,}800$ lbs. The allowable stresses are given in § 33, "Specifications for Steel Mine Structures," Appendix I, and are as follows, in lbs. per sq. in.:

Tension	16,000
Compression	$16{,}000-70\frac{l}{r}$

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where l = length of the member in inches, and r = the radius of gyration of the member in inches.

Rivets and pins, bearing	22,000
Rivets and pins, shear	11,000
Pins, bending on extreme fiber	24,000
Plate girder webs, shear on net section	10,000
Bearing on 1-2-4 Portland cement concrete	500

Design of Compression Members.—*Member X-1.* Stress = + 14,400 lbs.

The upper chord will be made of two angles with unequal legs, placed back to back, with the shorter legs turned out and separated by $\frac{3}{8}$ in. gusset or connection plates.

Try two $3\frac{1}{2}'' \times 3'' \times \frac{5}{16}''$ angles. From table on page 196 Cambria (edition 1909), or page 145 Carnegie (edition 1903), the least radius of gyration, r, is 1.10 in. The unsupported length of the member is 11 ft. 6 in. = 138 in., and $l/r = 138 \div 1.10 = 125$. The allowable ratio of l/r is 125 (Specifications, § 100), so that the member just satisfies this specification. The allowable stress per sq. in. = 16,000 — $70l/r$ = 16,000 — 8,750 = 7,250 lbs. The area required will be 14,400/7,250 = 1.99 sq. in. The area of two angles $3\frac{1}{2}'' \times 3'' \times \frac{5}{16}''$ = 3.88 sq. in. The area is greater than required to take the stress but the section is required by the specifications for ratio of length to radius of gyration. To make the two angles act together as one piece, it is necessary to rivet them together at intervals, such that the two angles acting singly will be stronger than the two angles acting together. On page 180 Cambria, or page 113 Carnegie, the least radius of gyration of a $3\frac{1}{2}'' \times 3'' \times \frac{5}{16}''$ angle about a diagonal axis is $r = 0.63$ in. The angles must, therefore, be riveted together not farther apart than $0.63 \times 125 = 78.8$ in. It is the common practice to rivet angles in compression about every $2\frac{1}{2}$ to 3 ft.

The truss will be shipped in two parts and in order to avoid a splice, and because the difference in the stresses is small, the entire top chord will be made of two angles $3\frac{1}{2}'' \times 3'' \times \frac{5}{16}''$.

Member 1-2.—Stress = + 4,200 lbs.

The length is 5 ft. 9 in. = 69 in. The member must have a radius of gyration not less than $r = 69/125 = 0.55$. From page 195 Cambria, the radius of gyration of two angles $2'' \times 2'' \times \frac{1}{4}''$ is $r = 0.61$, or from page 174 Cambria, the least radius of gyration of one angle $3'' \times 3''$

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$\times \frac{1}{4}''$ is $r = 0.59$. The area of 2 angles $2'' \times 2'' \times \frac{1}{4}''$ is equal to 1.88 sq. in., and the area of one angle $3'' \times 3'' \times \frac{1}{4}''$ is equal to 1.44 sq. in. Where one angle is used for a strut it should be fastened by both legs

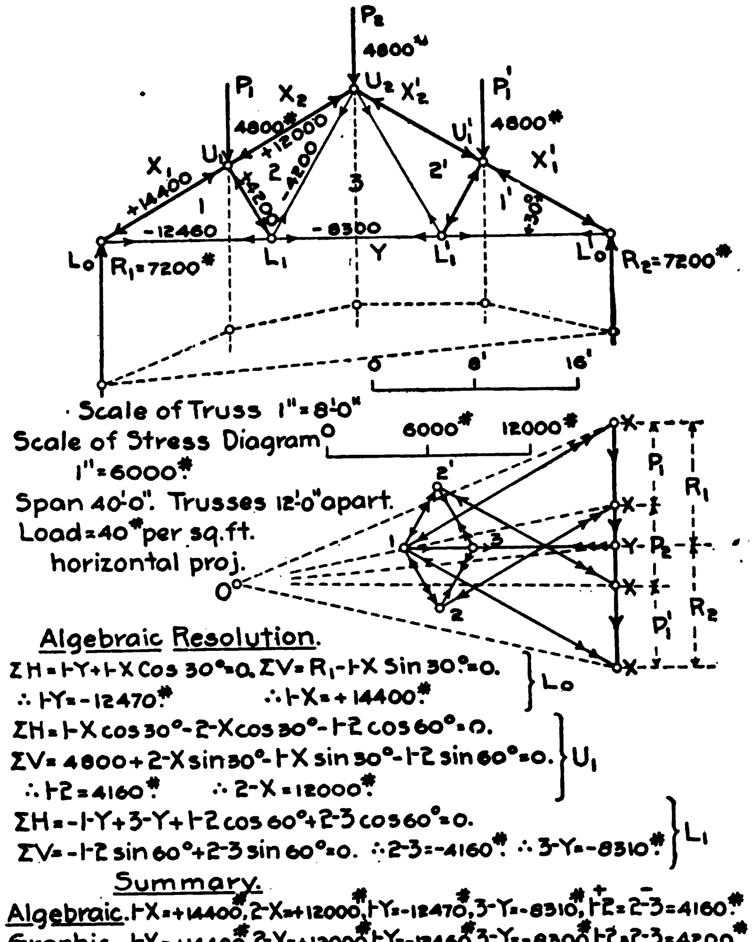

FIG. 156 b. STRESSES IN A ROOF TRUSS.

or only one leg may be taken as effective (Specifications, § 39). The allowable stress for the one angle $3'' \times 3'' \times \frac{1}{4}''$ is $= 16{,}000 - 70 \times 69/0.59 = 16{,}000 - 8{,}200 = 7{,}800$ lbs. per sq. in. The required area is $4{,}800/7{,}800 = 0.62$ sq. in. The single angle need be fastened by but

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one leg. The two angles $2'' \times 2'' \times \frac{1}{4}''$ make the best section and will be used.

Design of Tension Members.—*Member 1–Y.* Stress $= -$ 12,460 lbs.

The net area required is $12,460/16,000 = 0.78$ sq. in. The length of the member is 13 ft. 4 in. $=$ 160 in. From Specifications, § 101, the ratio of l/r must not exceed 200. The least radius of gyration of the section will then be $r = 160/200 = 0.80$. From page 196 Cambria, two angles $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ with the long legs placed together have a minimum radius of gyration, $r = 0.96$.

The gross area of the section must be such that there will be a net area of not less than 0.78 sq. in. after the area of the rivet holes in any section has been deducted Try two angles $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$. It will be necessary to deduct the area of one rivet hole from each angle. The diameter of the rivet hole deducted is taken $\frac{1}{8}$ in. larger than the diameter of the rivet before driving. Assuming $\frac{5}{8}$ in. rivets it will be necessary to deduct 0.19 sq. in. from each angle (Table XLVI). The net area of the two angles is $2.63 - 0.38 = 2.25$ sq. in. The area is more than sufficient to take the stress, but the member just satisfies the specification for length, and will be used.

Member 3–Y.—Stress $= -$ 8,300 lbs.

The length of the member is 13 ft. 4 in. $=$ 160 in., which makes it necessary to use the same section as was used for member *1–Y*. The member will be made of two angles $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ with long legs placed together.

Design of Member 2–3.—Stress $= -$ 4,200 lbs.

The length of the member is 13 ft. 4 in. $=$ 160 in., which makes it necessary to use the same section as was used for member *1–Y*. The member will be made of two angles $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ with long legs placed together.

DESIGN OF STEEL FRAME BUILDINGS. Framework.—The framework of a steel frame building consists of a series of transverse bents, which carry the purlins on the tops of the trusses, and girts on the sides of the posts to carry the covering, Fig. 157. The framework is braced by diagonal bracing in the planes of the roof and the sides of the building, and in the plane of the lower chords. A transverse bent consists of a roof truss supported at the ends on columns and is braced against endwise movement by means of knee braces. The framing plan for a steel frame building is shown in Fig. 157.

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Transverse Bents.—A number of the common forms of transverse bents are shown in Fig. 158. Transverse bents (*a*), (*b*), (*d*) and (*h*) are used for boiler houses, shops, etc., while (*e*), (*f*), (*g*) and (*c*) are used when side sheds are desired.

Columns.—The common forms of columns used in steel frame buildings are shown in Fig. 159. Column (*f*), 4 angles and 1 plate, and column (*g*), 4 angles laced are commonly used for transverse bents. The best corner column is a single angle as shown in (*i*). Columns (*a*) to (*e*) are used to support heavy loads.

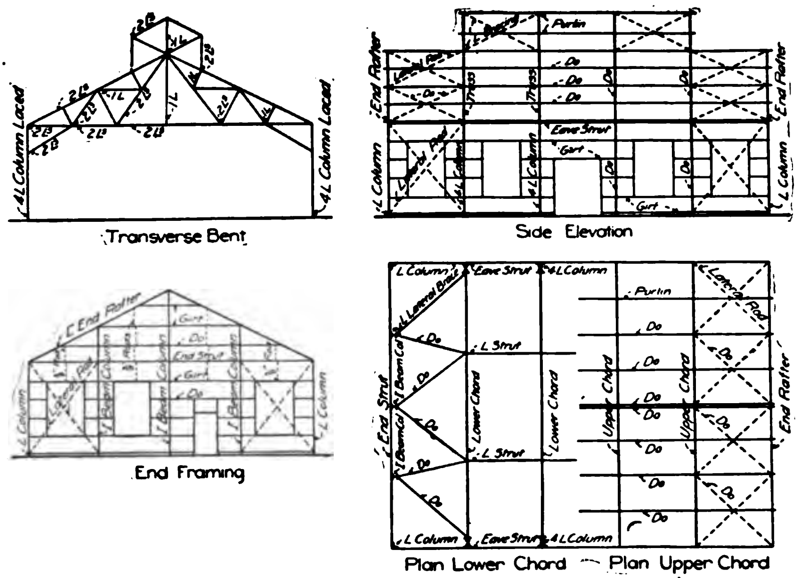

FIG. 157.

A Steel Transformer Building.—The framework of a steel frame transformer building is shown in Fig. 160 and Fig. 161. The trusses are Fink trusses with the members made of angles placed back to back. The main columns carrying the roof trusses are made of four angles laced, the section being I-shaped, each flange being composed of two angles placed back to back with the long legs outstanding, and the web consisting of lacing. The columns in the end of the building are made of 9″ I-beams. The main purlins are made of 5″ channels @ 6½ lbs., while the girts are 4″ channels @ 5¼ lbs. The purlins are spaced less

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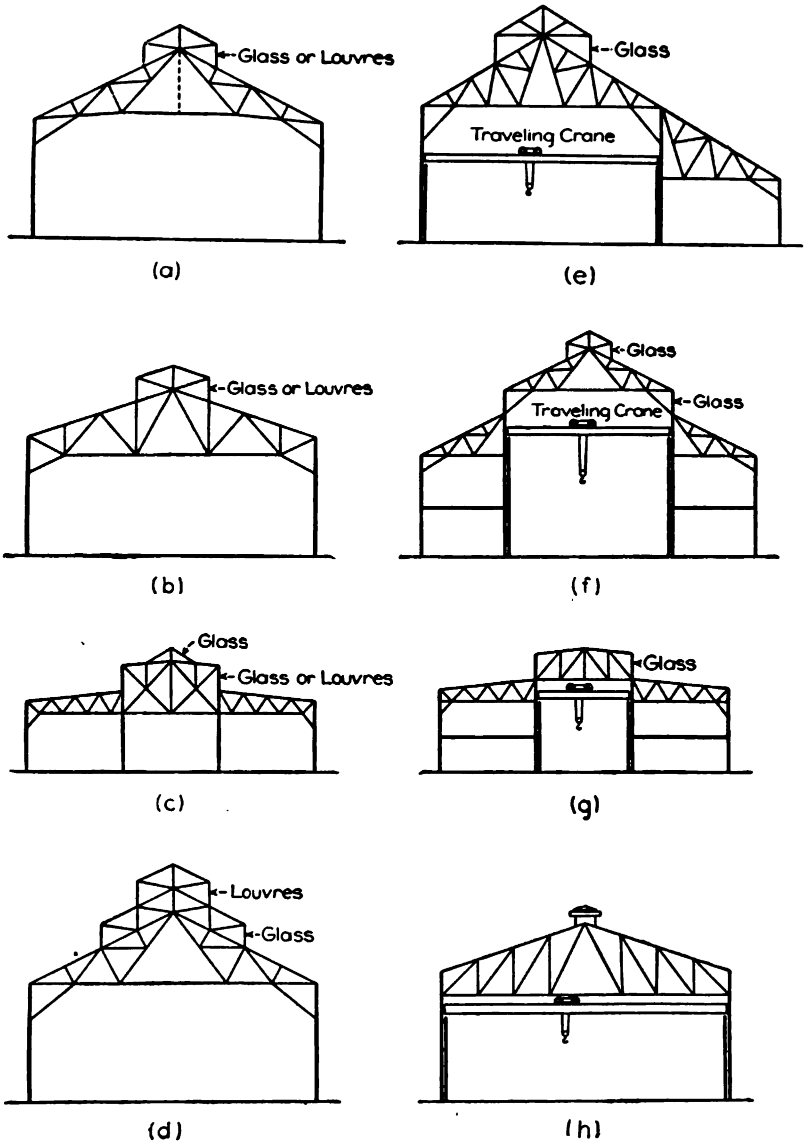

FIG. 158. TYPES OF TRANSVERSE BENTS.

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than 4 ft. 9 in., which is a maximum spacing where corrugated steel roofing is used without sheathing. The steel framework is braced in the plane of the top chord and the sides and ends of the building by means of diagonal rods ⅞ in. in diameter. The crane girder beams in

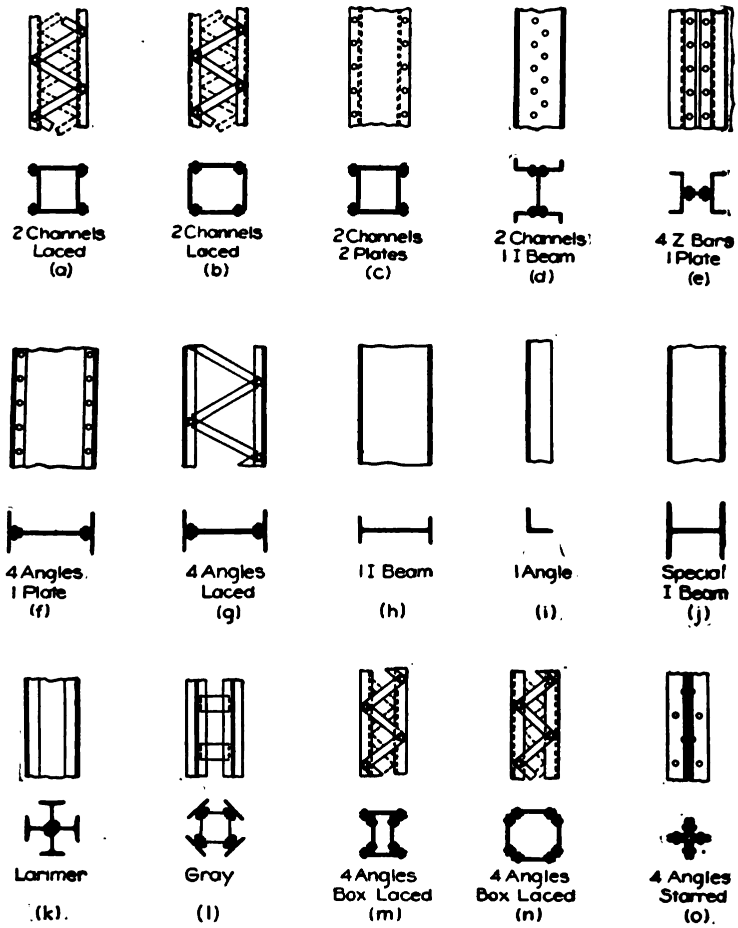

FIG. 159. TYPES OF MILL BUILDING COLUMNS.

the plane of the lower cord brace the building longitudinally, the diagonal bracing being composed of angles.

Corrugated Steel Covering.—The plans for the corrugated steel covering on the roof and sides are shown in Fig. 162 and Fig. 163.

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The corrugated steel for the roof is No. 22 gage steel with 2½ in. corrugations, while the corrugated steel for the sides is No. 24 gage steel with 2½ in. corrugations. The flashing and ridge roll are made of No. 22 flat sheet steel.

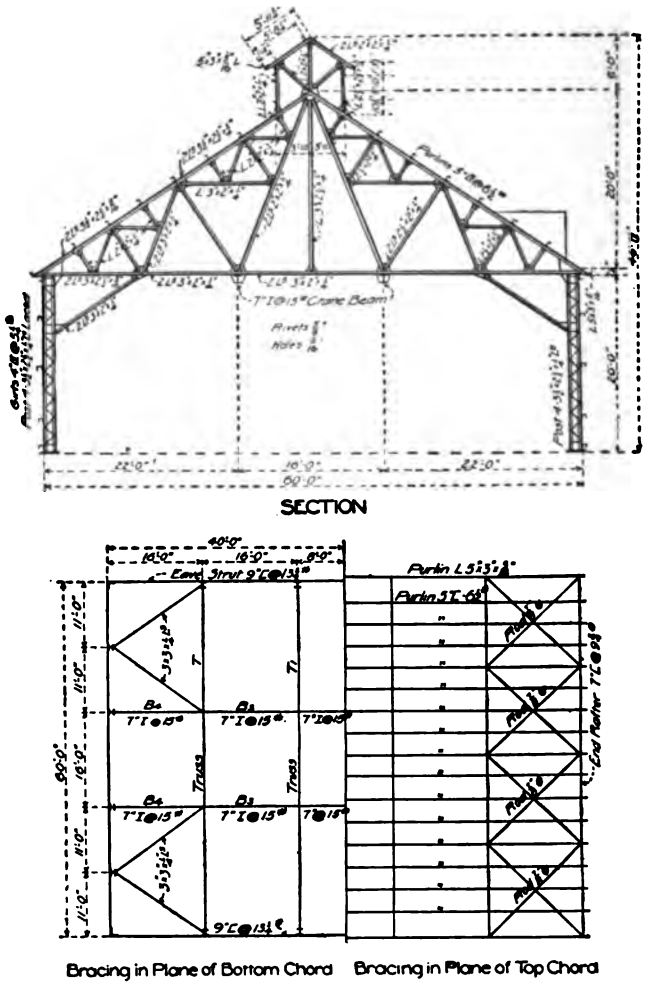

FIG. 160. PLANS OF A TRANSFORMER BUILDING.

To prevent the condensation of moisture on the inside of the steel roof and the resulting dripping, anti-condensation lining was used, as is shown in Fig. 163. This lining was constructed as follows: galvan-

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ized wire poultry netting was fastened to one eave purlin, was passed over the ridge, stretched tight and fastened to the other eave purlin.

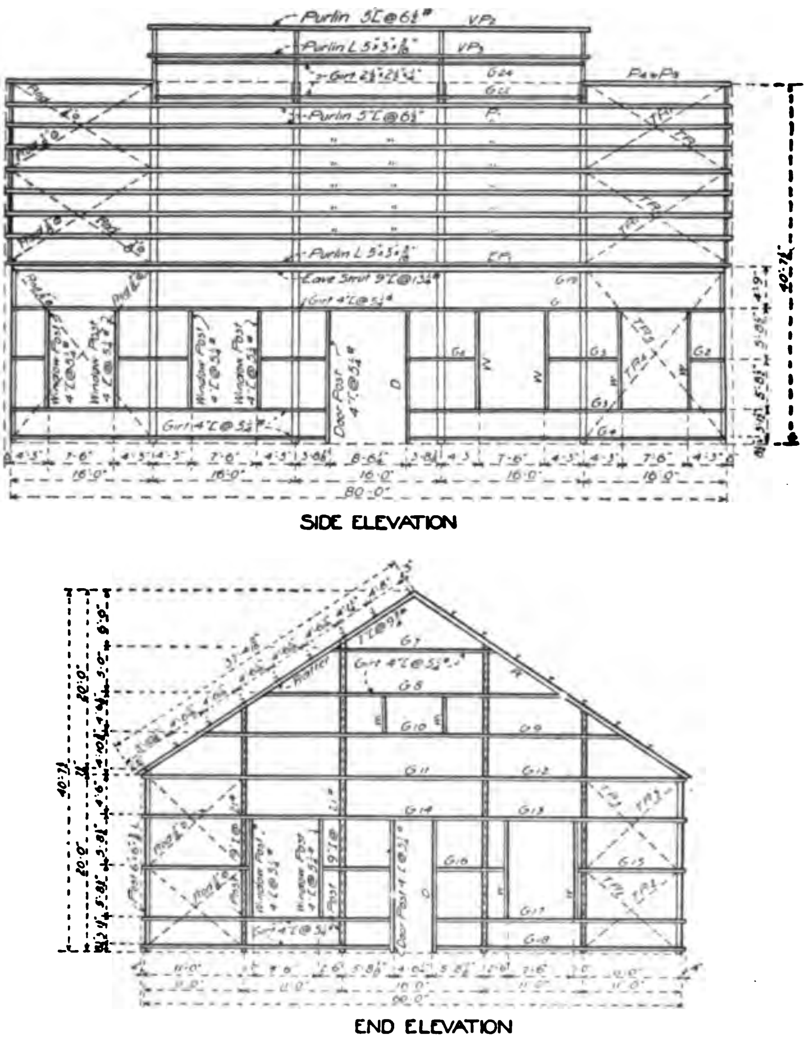

FIG. 161. PLANS OF A TRANSFORMER BUILDING.

The edges of the wire were woven together by means of wire clips. On the wire netting was laid two layers of asbestos paper $\frac{1}{16}$ in. thick

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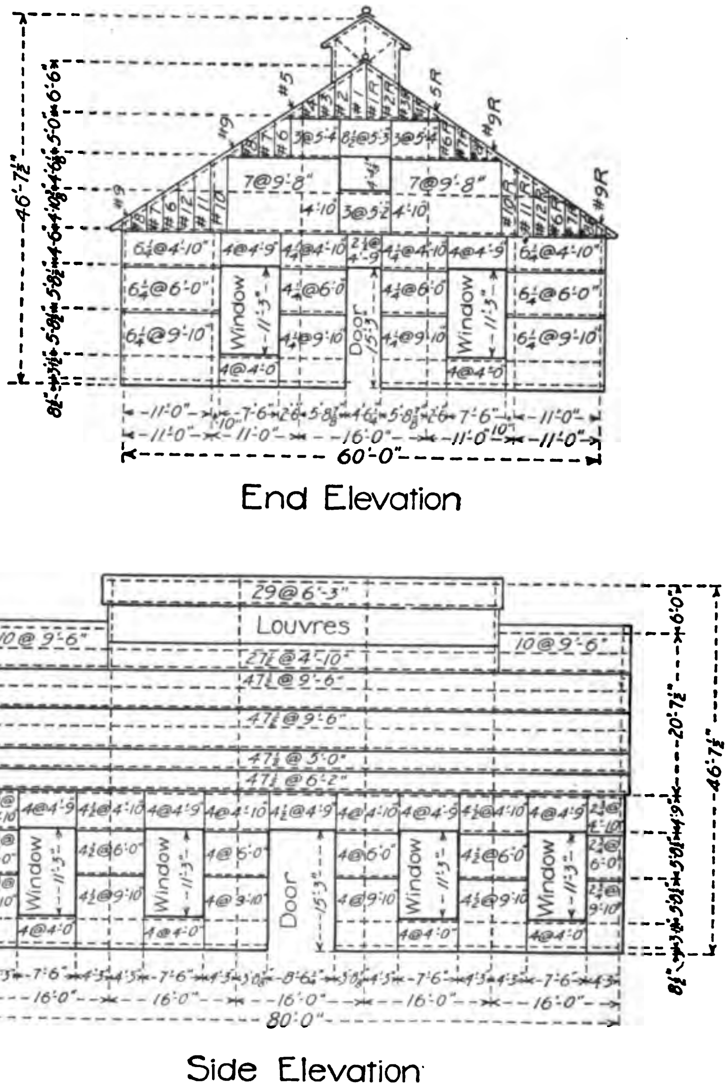

FIG. 162. CORRUGATED STEEL PLANS FOR TRANSFORMER BUILDING.

and on top of the asbestos was laid two layers of tar paper. The corrugated steel was then laid on top of the roof in the usual way and was fastened to the purlins by means of long soft iron wire nails spaced

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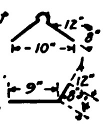
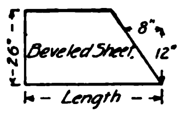
Corrugated Steel List for Building

Rectangular Sheets				Beveled Sheets as per Sketch				
No.	U.S.S.G.	Length	Marks	No.	U.S.S.G.	Length	Marks	
55	#22	4'-10"		4	#24	7'-1½"	2# 1	2# 1R
95	"	5'-0"		4	"	5'-9½"	2# 2	2# 2R
95	"	6'-2"		4	"	4'-5½"	2# 3	2# 3R
58	"	6'-3"		4	"	3'-1½"	2# 4	2# 4R
40	"	9'-6"		4	"	1'-9½"	2# 5	2# 5R
190	"	9'-6"		8	"	6'-0"	4# 6	4# 6R
48	#24	4'-0"		8	"	4'-8"	4# 7	4# 7R
62	"	4'-9"		8	"	3'-4"	4# 8	4# 8R
87	"	4'-10"		8	"	2'-0"	4# 9	4# 9R
7	"	5'-2"		4	"	10'-0"	2# 10	2# 10R
7	"	5'-3"		4	"	8'-8"	2# 11	2# 11R
12	"	5'-4"		4	"	7'-4	2# 12	2# 12R
87	"	6'-0"						
28	"	9'-8"						
87	"	9'-10"						

84 lineal feet Ridge Roll #22 Flat Steel.
100 lineal feet Flashing #22 Flat Steel

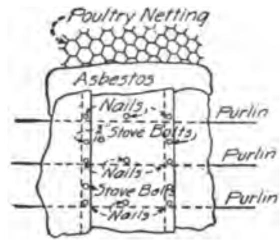




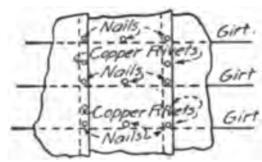
All sheet steel Painted one coat Red Lead.
Sheets 26" wide.
Corrugations 2½"

55 squares Asbestos.
1300 lin. ft. 60" Poultry Netting

Corrugated Steel on Sides, No. 24 Black, Painted, 1 Corrugation Side lap and 4" End lap.
Corrugated Steel on Roof, No. 22 Black, Painted, 2 Corrugations Side lap and 6" End lap



Method of Fastening Steel and Lining on Roof



Method of Fastening Steel on the Sides

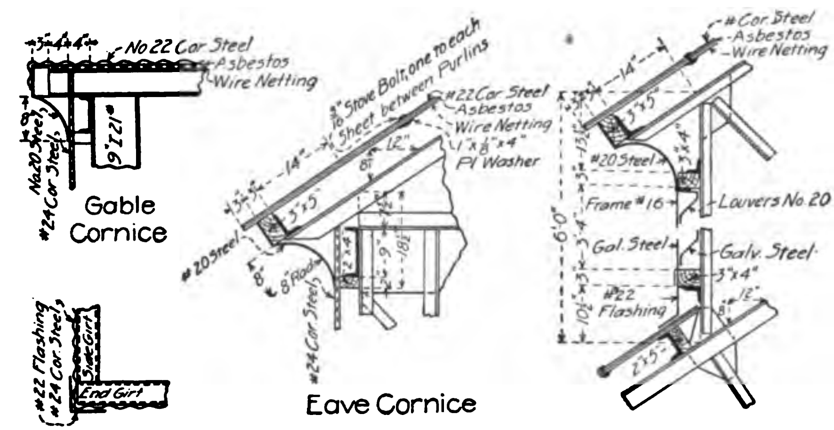


FIG. 163. CORRUGATED STEEL LIST AND DETAILS FOR TRANSFORMER BUILDING.

as shown in Fig. 163. To prevent the lining from sagging, stove bolts $\frac{3}{16}$ in. in diameter with $1'' \times \frac{1}{8}'' \times 4''$ flat washers on the lower side were placed between the purlins. The author would recommend that

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purlins be spaced ½ the usual distance where anti-condensation lining is used and the stove bolts be omitted.

Corrugated steel comes in stock sizes in even feet from 4 to 10 ft. long. The minimum thickness of corrugated steel for roofing should be No. 22 gage, while No. 24 is the minimum gage to use for the sides. The common methods for fastening corrugated steel are shown in Fig. 164. Corrugated steel should be laid with a 6 in. end lap and two

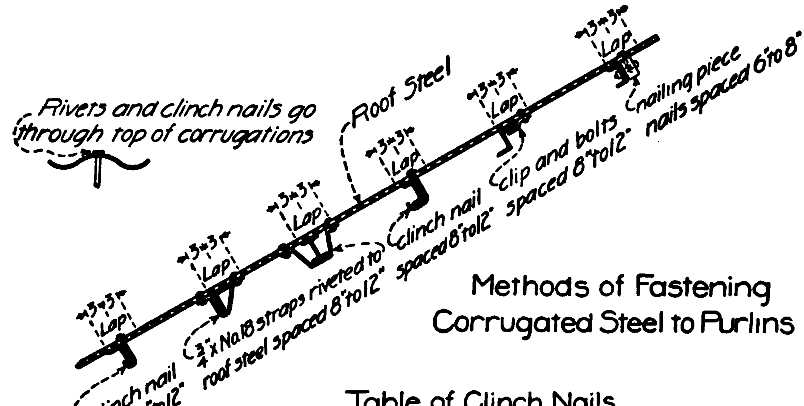

Table of Clinch Nails

L Purlin leg	3"	4"	5"	6"	7"
Length	5"	6"	7"	8"	9"
No. per lb.	32	29	23	21	18
C Purlin leg	3"	4"	5"	6"	7"
Length	6"	7" or 8"	9"	10"	11"
No. per lb.	29	21	18	16	14

FIG. 164. METHODS OF FASTENING CORRUGATED STEEL TO PURLINS AND GIRTS.

corrugations side lap on the roof, and with a 4 in. end lap and one corrugation side lap on the sides of the buildings. Where corrugated steel is used without sheathing the edges should be riveted together with copper or iron rivets at intervals of not less than 12 in. The safe load for corrugated steel for different spans is shown in Fig. 3, "Specifications for Steel Mine Structures," Appendix I. Corrugated steel covering may be laid on timber sheathing, to which it is fastened with barbed roofing nails.

Details for corrugated steel roofing, walls and finish are given in Fig. 165.

The roofs of steel frame buildings are also covered with tar and gravel roofing, slate, tile or some of the patented composition roofings. Specifications for tar and gravel roofs are given in the "Specifications

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for Steel Mine Structures" in Appendix I. For additional details of roofs see the author's "The Design of Steel Mill Buildings."

Steel Frame Building with Plaster Walls.—The steel frame building shown in Fig. 166 and Fig. 167 was covered with expanded metal and plaster, walls and roof constructed as follows: The side walls were made by fastening ¾ in. channels at 12 in centers to the steel framework and then covering this framework with expanded metal wired on.

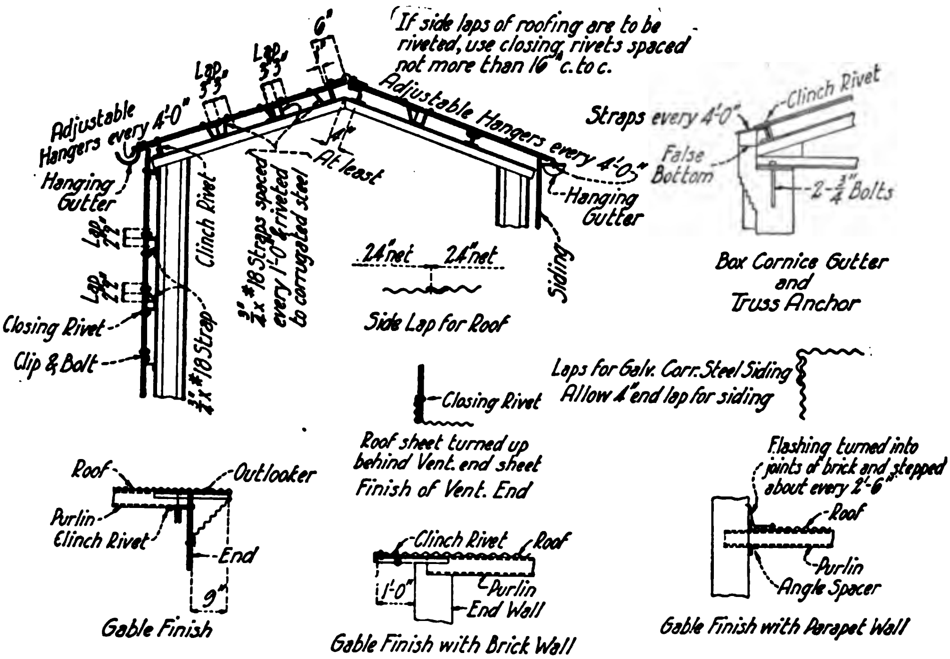

Fig. 165. Details for Corrugated Steel.

The expanded metal was then covered on the outside with a coating of cement mortar composed of one part Portland cement and two parts sand, and on the inside with a gypsum plaster, making the walls about 2 in. thick The roof consists of a 2½ in. concrete slab reinforced with expanded metal, this slab being covered with 10 in. × 12 in. slate nailed directly to the concrete.

For the details of a steel frame hoist house with plaster walls, see the description of the Tonopah-Belmont head frame in Chapter VI. Steel frame buildings may also be constructed with masonry filled walls.

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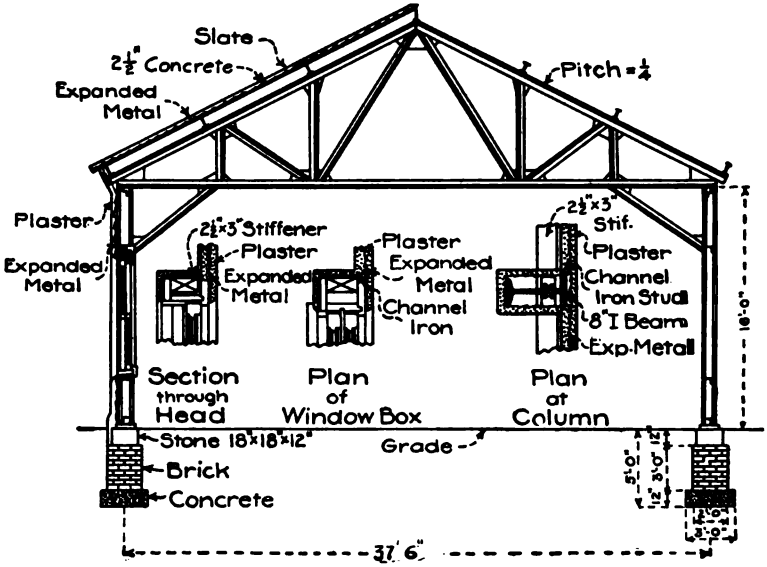

FIG. 166. CROSS-SECTION OF STEEL BUILDING COVERED WITH EXPANDED METAL AND PLASTER.

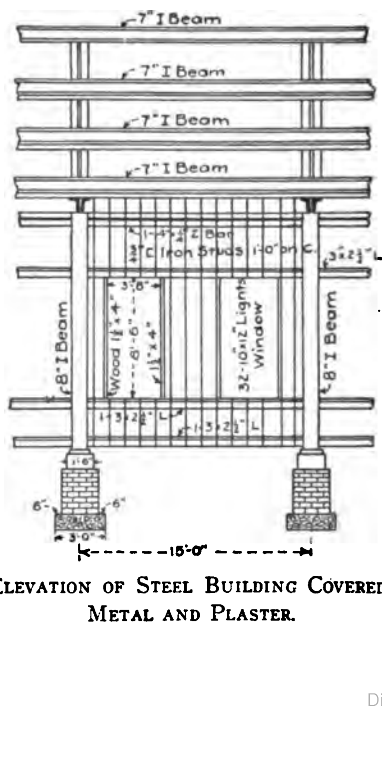

FIG. 167. SIDE ELEVATION OF STEEL BUILDING COVERED WITH EXPANDED METAL AND PLASTER.

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Windows and Sky Lights.—Mill and mine buildings should have an ample amount of glazing in the form of windows and sky lights. For ordinary windows double strength glass gives very satisfactory results. For sky lights and where windows are liable to be broken, wire glass should be used. The best glass for glazing windows in

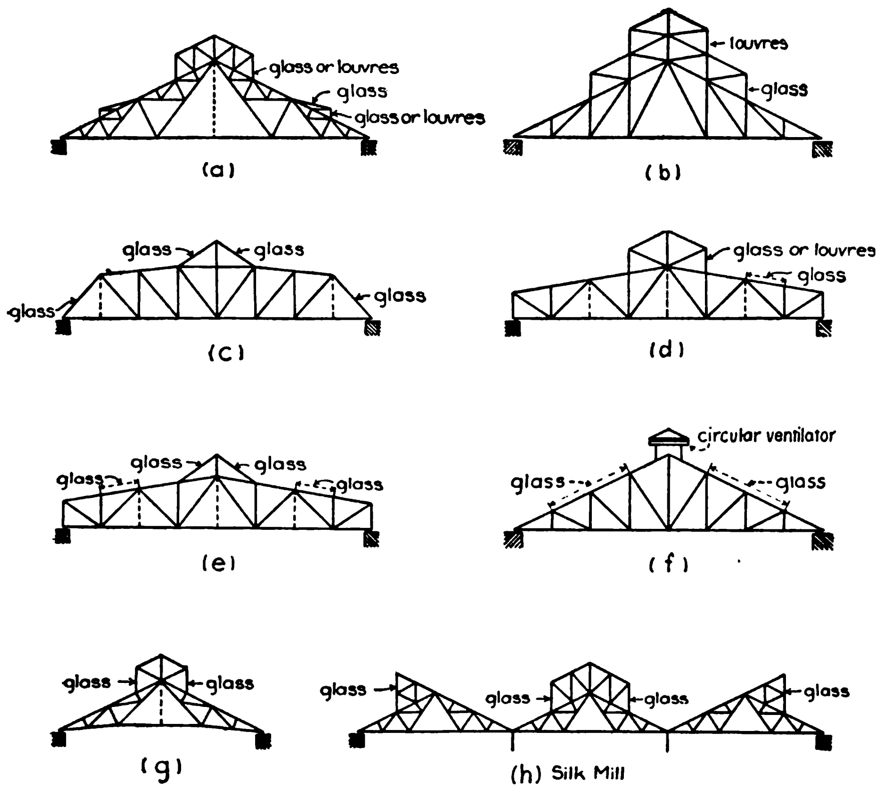

FIG. 168. TYPES OF ROOFS FOR LIGHTING AND VENTILATION.

industrial plants is "factory ribbed glass" with twenty-one ribs to the inch, the ribs being placed on the inside of the window. This glass is considerably more expensive than plain glass but is much more satisfactory.

Translucent fabric made by imbedding wire cloth in a translucent material made of linseed oil, is also used for glazing in industrial buildings. Translucent fabric will be charred by a live coal but is prac-

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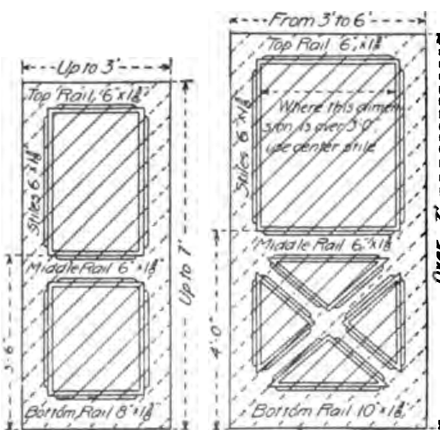
tically fire-proof. It shuts off part of the light, making it possible for men to work under it without shading.

The amount of glazed surface required in mill buildings depends upon the use to which the building is put, the material used in glazing, the location and the angle of the windows and sky lights, and the clearness of the atmosphere. It is common to specify that not less than 10 per cent of the exterior surface of mill buildings and 25 per cent of the exterior surface of machine shops should be glazed.

Several different types of trusses with provision for lighting and ventilation are given in Fig. 168. For additional data, together with details of windows and sky lights, see the author's "The Design of Steel Mill Buildings."

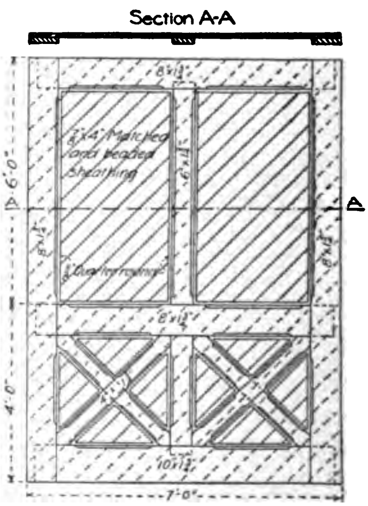
Ventilators.—Mill buildings may be ventilated by means of monitor ventilators or by means of circular ventilators. The sides of the monitor ventilator in Fig. 160 was fitted with louvres which were to be closed in cold weather. Buildings of this type should have glazed sash so that when the ventilators are closed the light will not be cut off. For details of ventilators see the author's "The Design of Steel Mill Buildings."

Wooden Doors.—Wooden doors are usually constructed of matched pine sheathing nailed to a wooden frame as shown in Fig. 169 and Fig. 170.



Swing Wooden Doors

FIG. 169.



Sliding Wooden Door

FIG. 170.

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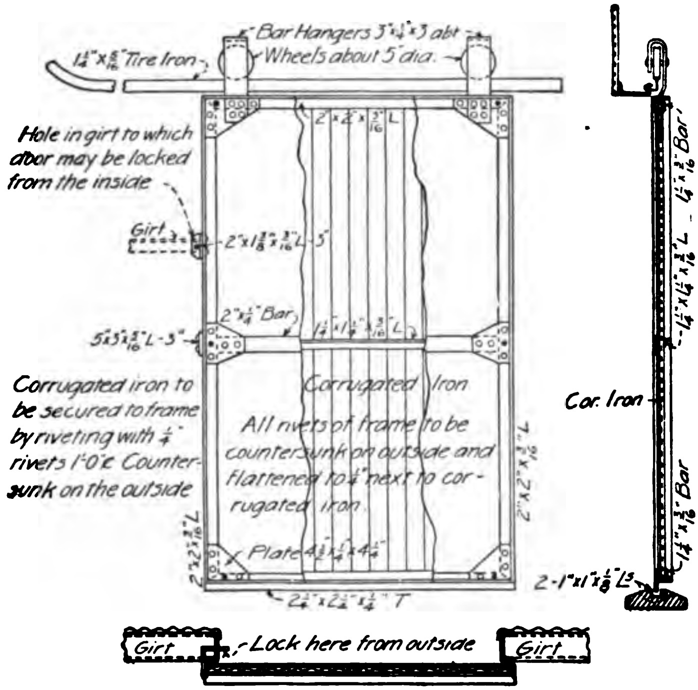
Designs for wooden swing doors are shown in Fig. 169, and for a wooden sliding door in Fig. 170. These doors are made of white pine. Doors up to four feet in width should be swung on hinges; wider doors should be made to slide on an overhead track or should be counterbalanced and raise vertically. Sliding doors should be at least 4 in. wider and 2 in. higher than the clear opening.



Steel Sliding Door

FIG. 171. DETAILS OF STEEL SLIDING DOOR.

"Sandwich" doors are made by covering a wooden frame with flat or corrugated steel. The wooden framework of these doors is commonly made of two or more thicknesses of ⅞ in. dressed and matched white pine sheathing not over 4 in. wide, laid diagonally and nailed with clinch nails. Care must be used in handling sandwich doors made as above or they will warp out of shape. Corrugated steel with 1¼ in. corrugations makes the neatest covering for sandwich doors.

For swing doors use hinges about as follows: For doors 3 ft. × 6 ft.

or less use 10 in. strap or 10 in. T-hinges; for doors 3 ft. × 6 ft. to 3 ft. × 8 ft. use 16 in. strap or 16 in. T-hinges; for doors 3 ft. × 8 ft. to 4 ft. × 10 ft. use 24 in. strap hinges.

Steel Doors.—Details of a steel sliding door are shown in Fig. 171. Steel doors should be covered with corrugated steel, preferably with 1¼ in. corrugations.

Floors.—Floors for mill buildings may be made (1) of concrete laid either on a solid foundation or in the form of slabs reinforced for temperature and contraction stresses; (2) timber on a solid cement or tar concrete base, or (3) may be made of cinders or other material. Concrete floors are ordinarily laid on a solid foundation of gravel or cinders. The main body of the floor is made of concrete 2½ in. to 3 in. thick. The concrete should be mixed in the proportions of one part first-class Portland cement, two parts clean, sharp, coarse sand and four parts broken stone or gravel. The broken stone or gravel should have no particles that will not pass a 1½ in. ring. The concrete should be mixed very thoroughly and should be deposited in place and be thoroughly tamped until moisture rises to the surface. Before the base is set the wearing surface, consisting of one part Portland cement and two parts clean, sharp, coarse sand, and ¾ to 1 in. thick, should be applied and troweled smooth. To get a glazed finish (which is not desirable) the surface is rubbed during the time of setting, the glaze being formed by adding dry cement to the top as the rubbing proceeds. Care should be used to have the top surface of the concrete clean of all material and free from water before placing the wearing surface.

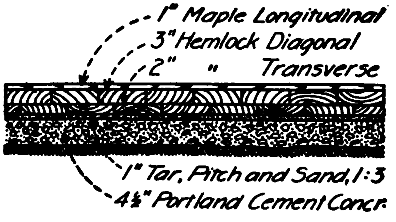


FIG. 172.

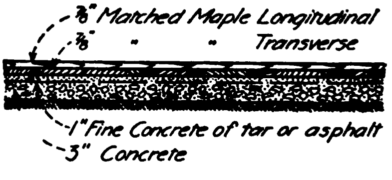


FIG. 173.

The most satisfactory floor for all purposes is a timber floor carried on a bituminous concrete base. This floor is ordinarily constructed as follows: After the subgrade has been thoroughly tamped, 3 in. × 3 in. timber joists spaced about 18″ centers are supported at the proper ele-

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vation by means of stakes driven in the subgrade. Bituminous concrete, made by mixing stone or gravel with a refined coal tar at a temperature of 200 to 300 degrees, is then deposited around the joists and is thoroughly tamped until the surface is slightly below the tops of the timber joists. On top of the timber joists is then laid a subfloor consisting of 2 in. pine planks fully spiked to the joists. The wearing surface is then placed transversely on top of the subfloor. The best wearing surface is ⅞ in. maple flooring, although hard pine floors give fairly satisfactory results. Specifications for concrete and timber floors are given in Appendix I. For details of timber shop floors see Fig. 172 and Fig. 173. Additional details of shop floors are given in the author's "The Design of Steel Mill Buildings."

FOUNDATIONS.—The allowable pressure on foundations depends upon local conditions, which should be carefully examined in every case. In the absence of definite information the pressures on foundations should not exceed the following in tons per sq. ft.

Soft clay and loam	1
Ordinary clay and dry sand mixed with clay	2
Dry sand and dry clay	3
Hard clay and firm, coarse sand	4
Firm, coarse sand and gravel	5
Shale rock	8
Hard rock	20

The pressure of column bases on Portland cement concrete should not exceed 500 lbs. per sq. in.; the pressure on good sandstone and limestone should not exceed 400 lbs. per sq. in., while the pressure on inferior masonry should be very much less than the above.

PAINT.—Oil paints are made by mixing a pigment with linseed oil. Red lead, graphite, iron oxide and carbon are used as pigments. Carbon may be used in the form of graphite or lamp black. The amount of pigment per gallon of oil varies as follows: red lead 15 to 33 lbs., iron oxide 15 to 20 lbs., graphite 3 to 12 lbs. The paint requiring the most pigment gives the thicker paint and therefore has a less covering capacity per gallon. Under ordinary conditions very satisfactory results are obtained with any of the above paints providing the materials are pure and thoroughly mixed, the surface is clean, dry and warm, and the painting is well done. However, where struc-

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tural steel is exposed to smelter gases, graphite and carbon paints appear to give better results than either red lead or iron oxide paints.

There are many patented paints on the market, most of them containing either graphite or asphalt. Several of these paints give very satisfactory results. For painting structural steel work exposed to smelter or other corrosive gases coal tar paint will be found very efficient. This coal tar paint is made by mixing Portland cement, kerosene and refined coal tar, in the proportions of 16 parts of refined coal tar, 4 parts Portland cement, and 3 parts kerosene oil. The kerosene oil and Portland cement are first thoroughly mixed and then immediately added to the coal tar. This makes a very efficient covering, as the Portland cement prevents the coal tar from melting in hot weather or cracking in cold weather. If it is later desired to paint structural steel work with an oil paint it will be necessary to scrape off the coal tar paint before applying the oil paint.

For data on paints and the cost of painting see Chapter XV; also see the author's " The Design of Steel Mill Buildings."

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CHAPTER X.

THE DESIGN OF BINS AND RETAINING WALLS.

Introduction.—Bins are designed to contain materials which are more or less nearly in a granular condition. Dry sand, screened anthracite coal, etc., are almost ideal semi-fluids, while bituminous coal and ashes are in a variable condition. Before considering the design of bins it will be necessary to briefly discuss the laws of semi-fluids and the design of retaining walls.

SEMI-FLUIDS.—The laws of perfect fluids and of solids have been studied experimentally and theoretically and are well known. Many fluids are perfect fluids only above certain temperatures, and as the temperature falls become "viscous fluids" and after passing the chill point become solids. Other materials such as dry sand and shot occupy an intermediate position between fluids and solids and are called "semi-fluids"; approaching fluids when water is added, and solids when a cementing material between the grains causes cohesion in the mass. Dry sand, shot, wheat and similar materials are almost perfect semi-fluids, the mass being without cohesion, the particles being held in place by friction on each other. The angle of internal friction is the angle whose tangent is the coefficient of internal friction of the particles upon one another, and is nearly always larger than the angle at which the material will stand if poured in a pile on a level floor. The angle of internal friction is commonly referred to as the angle of repose. If water be added to dry sand the angle of internal friction decreases and approaches zero, while if a cementing material such as clay be added, the angle of internal friction increases and approaches 90 degrees as a limit.

The laws of perfect semi-fluids such as sand and wheat have been studied experimentally and theoretically and are well known. Experiments made upon wheat in bins by the author and others and recorded in Chapter XVII, "The Design of Walls, Bins, and Grain Elevators," prove the following: (1) the horizontal pressures vary as the depth in very shallow bins; (2) in deep bins the horizontal pressure is less than the vertical pressure (0.3 to 0.6 of the vertical pressure, depending on

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the grain and many other conditions), and increases very little after a depth of $2\frac{1}{2}$ to 3 times the width or diameter of the bin is reached; (3) there is no active upward component in a granular mass; (4) the flow from an orifice in the side of a deep bin varies approximately as the cube of the orifice and is independent of the head, as long as the orifice is well covered.

PRESSURE ON RETAINING WALLS.—A retaining wall is a structure which sustains the lateral pressure of earth or some other granular mass which possesses some frictional stability. The pressure of the material supported will depend upon its condition, specific gravity, weight, and angle of repose or angle of internal friction. The pressure on a retaining wall varies as the depth, and as in the case of fluid pressure the center of pressure of the resultant thrust is at $\frac{1}{3}$ the height of the wall from the base.

Rankine's Formulas.—With a vertical wall and a horizontal surcharge, Fig. 176, the total resultant pressure is

$$P = \tfrac{1}{2} w \cdot h^2 \frac{1 - \sin\phi}{1 + \sin\phi} \tag{95}$$

where w is the weight of the filling in pounds per cubic foot, h is the depth of the wall in feet, ϕ is the angle of repose of the filling, and P

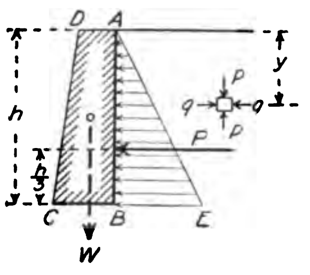

FIG. 176.

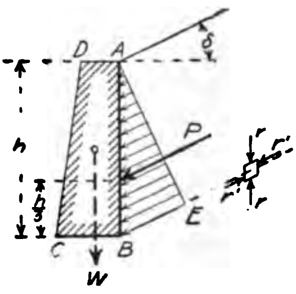

FIG. 177.

is the resultant pressure on the wall in pounds. The resultant pressure, P, will be horizontal.

For a vertical wall with surcharge at an angle δ, Fig. 177, the pressure is given by the formula

$$P = \tfrac{1}{2} w \cdot h^2 \cos\delta \frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}} \tag{96a}$$

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Where δ is equal to ϕ, formula (96a) becomes

$$P = \tfrac{1}{2} w \cdot h^2 \cos \phi \tag{96b}$$

The resultant pressure, P, is parallel to the inclined top surface.

Inclined Retaining Wall.—The pressure on an inclined retaining wall may be calculated by means of the ellipse of stress—see the author's "The Design of Walls, Bins and Grain Elevators." The pressure on an inclined retaining wall may also be calculated by means of the graphic solution shown in Fig. 179 if the direction of the thrust

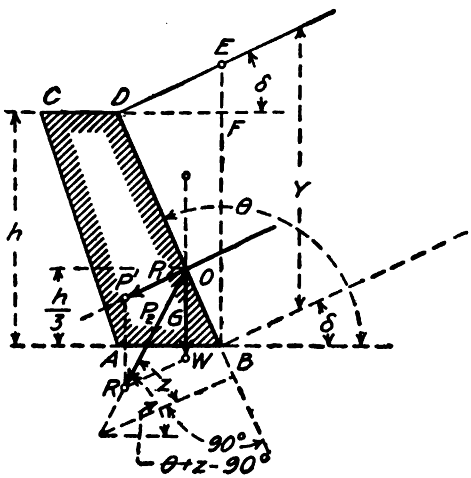

FIG. 178. PRESSURE ON AN INCLINED RETAINING WALL.

be known. From Rankine's theory we know that the resultant pressure on a vertical retaining wall is always parallel to the top surface where the surcharge is level or is inclined upwards away from the wall. The pressure on a retaining wall inclined away from the filling may then be calculated as follows:

In Fig. 178 the retaining wall $ACDB$ sustains the pressure of a filling having an angle of repose ϕ, and sloping up away from the top of the wall at an angle δ. Calculate P' the pressure on the plane E–B by means of formula (96a). P' acts at a point $\frac{1}{3}EB$ above B and is parallel to the top surface DE. Let the weight of the triangle of filling DBE be G, which acts through the center of gravity of the triangle and intersects P' at point O. Then P_2, the resultant of P' and G, will be the resultant pressure at O, and makes an angle z with a normal to the back of the wall, and an angle $\theta + z - 90°$ with the horizontal.

Graphic Method.—If the angle z, the angle between the back of the wall and a normal to the wall, is known, the resultant pressure on a wall may be calculated by a graphic method, Fig. 179, based on the

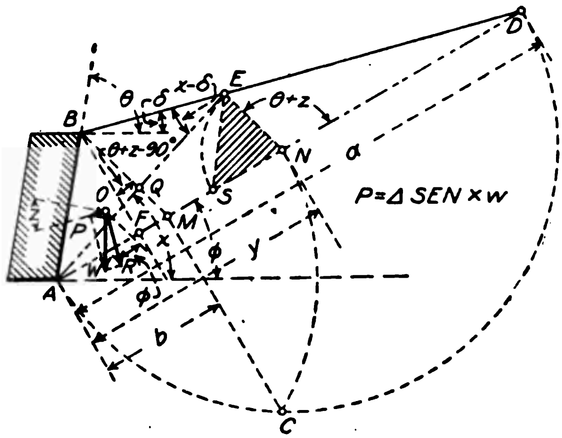

Fig. 179. Graphic Solution.

"theory of a wedge of maximum thrust." The graphic method will be described—the proof of the method is given in "The Design of Walls, Bins and Grain Elevators."

In Fig. 179 the retaining wall AB sustains the pressure of the filling with a surcharge δ and an angle of repose ϕ. It is required to calculate the resultant pressure P.

The graphic solution is as follows: Through B in Fig. 179 draw BM making an angle with BF, the normal to AD, equal to $\lambda = \theta + z - 90°$, the angle that P makes with the horizontal. With diameter AD describe arc ACD. Draw MC normal to AD and with A as a center and a radius AC describe arc CN. Then $AN = y$, $AM = b$ and $y = \sqrt{a \cdot b}$. Draw EN parallel to BM. With N as a center and radius EN, describe arc ES. Then AE is the trace of the plane of rupture, and $P =$ area $SEN \cdot w$.

Wall with a Loaded Filling.—In Fig. 180 the wall is loaded with a uniformly distributed load. Calculate h_1 by dividing the loading per square foot by w. Let $h + h_1 = H$. Then the resultant pressure, P, and the point of application of the resultant thrust will be as shown in Fig. 180, in which K is a coefficient depending upon angle of repose, ϕ, of the material, the angle θ, and the surcharge δ, formula (97).

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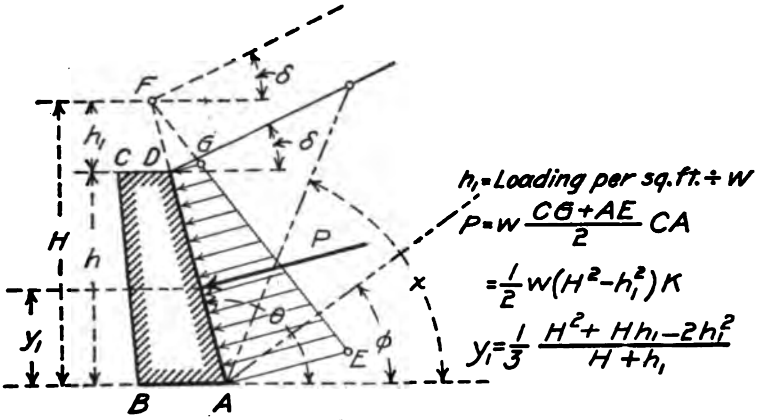

FIG. 180.

DESIGN OF MASONRY RETAINING WALLS. Center of Pressure.—It will be seen that for all cases the resultant pressure on the back of the wall will be given by the formula

$$P = \tfrac{1}{2} w \cdot h^2 \cdot K \tag{97}$$

Where K is the ratio of the horizontal to the vertical pressure, and is independent of the weight of the filling or the height of the wall, and depends upon the inclination of the back of the wall, θ, the angle of repose, ϕ, the angle of surcharge, δ, and the angle that the resultant thrust makes with a normal to the back of the wall, z. For any particular case, K will be a constant and the unit pressure at any point will vary as the height h. In Fig. 181 the resultant pressure, P, is represented by the area of the shaded triangles, and the center of pressure will be at $\frac{1}{3}h$ from the base, the same as for fluid pressure.

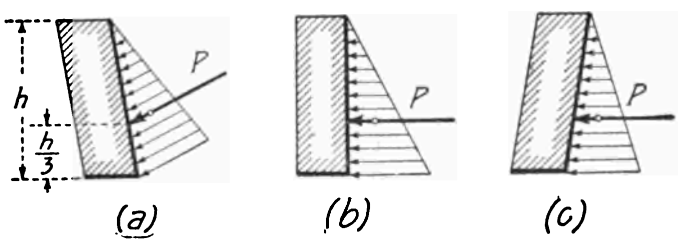

FIG. 181.

Stability of Retaining Walls.—A retaining wall must be stable (1) against overturning, (2) against sliding, and (3) against crushing the masonry or the foundation.

The factor of safety of a retaining wall is the ratio of the weight of a filling having the same angle of internal friction that will just cause failure to the actual weight of the filling For a factor of safety of 2 the wall would just be on the point of failure with a filling weighing twice that for which the wall is built.

1. *Overturning.*—In Fig. 182, let P, represented by OP', be the resultant pressure of the earth, and W, represented by OW, be the weight of the wall acting through its center of gravity. Then E, represented by OR, will be the resultant pressure tending to overturn the wall.

Draw OS through the point A. For this condition the wall will be just on the point of overturning, and the factor of safety against overturning will be unity. The factor of safety for $E=OR$ will be

$$f_0 = SW/RW \tag{98}$$

2. *Sliding.*—In Fig. 182 construct the angle $H1G$ equal to ϕ', the angle of friction of the masonry on the foundation. Now if E passes through 1, and takes the direction OQ, the wall will be on the point of sliding, and the factor of safety against sliding, f_s, will be unity. For $E=OR$, the factor of safety against sliding will be

$$f_s = QM'/RM \tag{99a}$$

Retaining walls seldom fail by sliding.

The factor of safety against sliding is sometimes given as

$$f_s = \frac{F}{H}\tan\phi'. \tag{99b}$$

where H is the horizontal component of P. Equations (99a) and (99b) give the same values only where the resultant P is horizontal.

3. *Crushing.*—In Fig. 182 the load on the foundation will be due to a vertical force F, which produces a uniform stress $p_1 = F/d$ over the area of the base, and a bending moment $= F\cdot b$, which produces compression, p_2, on the front and tension, p_2, on the back of the foundation. The sum of the tensile stresses due to bending must equal the sum of the compressive stresses, $=\frac{1}{4}p_2 d$. These stresses act as a couple

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through the centers of gravity of the stress triangles on each side, and the resisting moment is

$$M' = \tfrac{1}{4}p_2 \cdot d \cdot \tfrac{2}{3}d = \tfrac{1}{6}p_2 \cdot d^2 \qquad (a)$$

But the resisting movement equals the overturning moment and

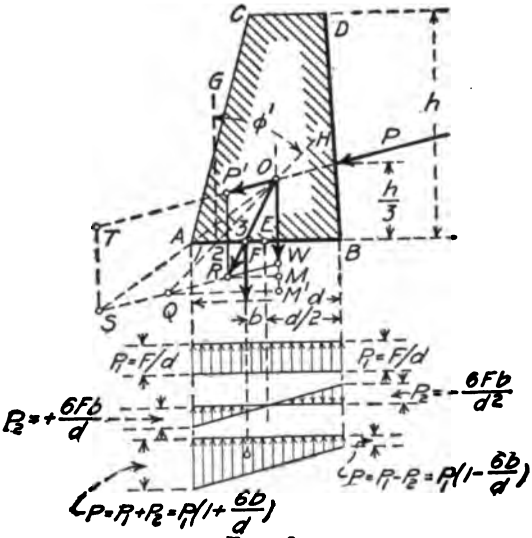

FIG. 182.

$$\tfrac{1}{6}p_2 \cdot d^2 = F\ b,$$

and

$$p_2 = \pm \frac{6F \cdot b}{d^2} \qquad (b)$$

The total stress on the foundation then is

$$p = p_1 \pm p_2 = p_1(1 \pm 6b/d) \qquad (99\text{c})$$

Now if $b = \tfrac{1}{6}d$, we will have

$$p = 2p_1, \text{ or } 0.$$

In order therefore that there be no tension, or that the compression never exceed twice the average stress, the resultant should never strike outside the middle third of the base.

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If the resultant strikes outside of the middle third of a wall in which the masonry can take no tension, the load will all be taken by compression and can be calculated as follows:

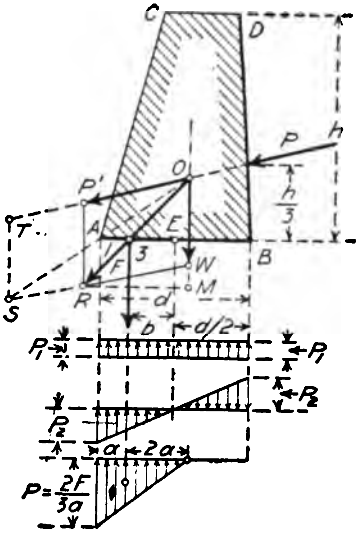

FIG. 183.

In Fig. 183 the resultant F will pass through the center of gravity of the stress diagram, and will equal the area of the diagram.

$$F=\tfrac{3}{2}p\ a$$

and

$$p=2F/3a \tag{100}$$

which gives a larger value of p than would be given if the masonry could take tension.

General Principles of Design.—The overturning moment of a masonry retaining wall of gravity section depends upon the weight of the filling, the angle of internal friction of the filling, the surcharge, and the height and shape of the wall. The resisting moment depends upon the weight of the masonry, the width of the foundation, and the cross-section of the wall. The most economical section for a masonry retaining wall is obtained when the back slopes toward the filling. In cold localities, however, this form of section may be displaced by heaving due to the action of frost, and it is usual to build retaining walls

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with a slight batter forwards. The front of the wall is usually built with a batter of from ½ in. to 1 in. in 12 in. In order to keep the center of gravity of the wall back of the center of the base it is necessary to increase the width of the wall at the base by adding a projection to the front side. Where the wall is built on the line of a right of way it is sometimes necessary to increase the width of the base by putting the projection on the rear side, making an L-shaped wall. The weight of the filling upon the base and back of the wall adds to the stability of the wall. Where the wall is built to support an embankment expensive to excavate, it is often economical to make the wall L-shaped, with all the projection on the front side.

In calculating the thrust on retaining walls great care must be exercised in selecting the proper values of w and ϕ, and the conditions of surcharge It will be seen from the preceding discussion that the value of the thrust increases very rapidly as ϕ decreases, and as the surcharge increases. Where the wall is to sustain an embankment carrying a railroad track, buildings, or other loads, a proper allowance must be made for the surcharge.

The filling back of the wall should be deposited and tamped in approximately horizontal layers, or with layers sloping back from the wall; and a layer of sand, gravel or other porous material should be deposited between the filling and the wall, to drain the filling downwards. To insure drainage of the filling, drains should be provided back of the wall and on top of the footing, and "weep-holes" should be provided near the bottom of the wall at frequent intervals to allow the water to pass through the wall. With walls from 15 to 25 feet high, it is usual to use "weepers" 4 inches in diameter placed from 15 to 20 feet apart. The "weepers" should be connected with a longitudinal drain in front of the wall. The filling in front of the wall should also be carefully drained.

The permissible point at which the resultant thrust may strike the base of the foundation will depend upon the material upon which the retaining wall rests. When the foundation is solid rock or the wall is on piles driven to a good refusal, the resultant thrust may strike slightly outside the middle third with little danger to the stability of the wall. When the retaining wall, however, rests upon compressible material the resultant thrust should strike at or inside the center of the base. Where the resultant thrust strikes outside of the center of the base, any settlement of the wall will cause the top to tip forward, causing

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unsightly cracks and local failure in many cases, and total failure where the settlement is excessive Where extended footings are used it may be necessary to use some reinforcing steel to prevent a crack in the footing in line with the face of the wall.

Plain masonry walls should be built in sections, the length depending upon the height of the wall, the foundation and other conditions. Under usual conditions the length of the sections should not exceed 40 ft., 30 ft. sections being preferable, and in no case should the length of the section exceed about three times the height. Separate sections should be held in line and in elevation, either by grooves in the masonry or by means of short bars placed at intervals in the cross-section of the wall, fastened rigidly in one section and sliding freely in the other. The back of the expansion joints should be water-proofed with 3 or 4 layers of burlap and coal tar pitch. The burlap should be about 30 inches wide, and the pitch and the burlap should be applied as on tar and gravel roofs. The joints between the sections of a retaining wall on the front side should be from $\frac{1}{8}$ to $\frac{1}{4}$ of an inch in width, and should be formed by a V-shaped groove made of sheet steel and fastened to the forms while the concrete is being placed. Where there is danger of the water in the filling percolating through the wall or in an alkali country, the surface of the back of the wall should be coated with a water-proof coating. The most satisfactory water-proof coating known to the author is a coal tar paint made by mixing refined coal tar, Portland cement and kerosene in the proportions of 16 parts refined coal tar, 4 parts of Portland cement and 3 parts of kerosene oil. The Portland cement and kerosene should be mixed thoroughly and the coal tar then added. In cold weather the coal tar may be heated and additional kerosene added to take account of the evaporation. This paint not only covers the surface but combines with it, so that two or three coats are sometimes required. While the surface of the concrete should be dry, coal tar paint will adhere to moist or wet concrete. In building retaining walls in sections, the end of the finished section should be coated with coal tar paint to prevent the adhesion to the next section.

DESIGN OF RETAINING WALLS.—The design of masonry retaining walls will be illustrated by the design of the retaining walls for West Alameda Avenue Subway, taken from the author's "The Design of Walls, Bins and Grain Elevators," second edition.

Design of Retaining Walls for West Alameda Avenue Subway, Denver, Colorado.—The height of the walls varied from 8′ to 29′ 3″,

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while the foundation soil varied from a compact gravel to a mushy clay. The design of the maximum section, which rests on a compact gravel, will be given. The concrete was mixed in the proportion of 1 part Portland cement, 3 parts sand and 5 parts screened gravel. Crocker and Ketchum, Denver, Colo., were the consulting engineers, the author being in charge of the design. The wall is shown in Fig. 184.

The following assumptions were made: Weight of concrete, 150 lbs. per cu. ft.; weight of filling, $w = 100$ lbs per cu. ft.; angle of repose of filling, $1\frac{1}{2}:1$ ($\phi = 33° \ 40'$); surcharge, 600 lbs. per sq. ft., equivalent to 6 feet of filling; maximum load on foundation, 6,000 lbs. per sq. ft.

Solution.—After several trials the following dimensions were taken: Width of coping 2′ 6″, thickness of coping 1′ 6″, batter of face of wall $\frac{1}{2}''$ in 12″, batter of back of wall $3\frac{1}{2}''$ in 12″, width of base 15′ $2\frac{5}{8}''$ (ratio of base to height = 0.52), front projection of base 4′ 0″, other dimensions as shown in Fig. 184. The calculations were made for a section of the wall one foot in length.

The property back of the wall will probably be used for the storage of coal, etc., and it was assumed that the surcharge came even with the back edge of the footing of the wall. The resultant pressure of the filling on the plane A–2 was calculated by the graphic method of Fig. 179 and Fig. 180, and was found to be $P' = 17{,}290$ lbs. The weight of the filling in the wedge back of the wall is $W'' = 16{,}435$ lbs., acting through the center of gravity of the filling. The resultant of P' and W' is $P = 23{,}850$ lbs. = the resultant pressure of the filling on the back of the wall. The weight of the masonry is $W = 33{,}144$ lbs., acting through the center of gravity of the wall, and the resultant of P and W is $E = 52{,}510$ lbs. = the resultant pressure of the wall and the filling upon the foundation. The vertical component of E is $F = 49{,}580$ lbs., and cuts the foundation, $b = 2.1$ feet from the middle.

1. *Stability Against Overturning.*—The line OD in this case is nearly parallel to the line QW which brings the point S in Fig. 184 at a great distance from the point W. The factor of safety against overturning was calculated on the original drawing and found to be $f_0 > 25$.

2. *Stability Against Sliding.*—The coefficient of friction of the masonry on the footing will be assumed to be $\tan \phi' = 0.57$ and $\phi' = 30°$. Through O, Fig. 184, draw OQ, cutting the base of wall $5A$ at 6, and making an angle $\phi' = 30°$ with a vertical line through 6. Then the factor of safety against sliding will be

$$f_s = QM'/RM = 2.5$$

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This is ample as the resistance of the filling in front of the toe will increase the resistance against sliding.

3. *Stability Against Crushing.*—In Fig. 184 the direct pressure will be $p_1 = 49{,}580/15.21 = 3{,}220$ lbs. per sq. ft.

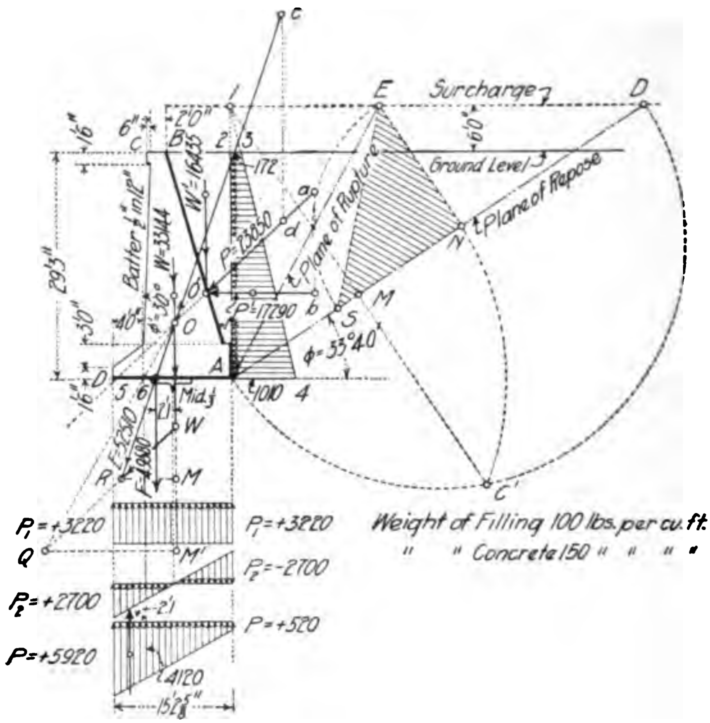

FIG. 184 RETAINING WALL, WEST ALAMEDA AVENUE SUBWAY.

The pressure due to bending will be

$p_2 = \pm 6F \cdot b/d^2 = \pm (6 \times 49{,}580 \times 2.1)/231.4 = \pm 2{,}700$ lbs. per sq. ft., and the maximum pressure is

$$p = 3{,}220 + 2{,}700 = +5{,}920 \text{ lbs. per sq. ft.}$$

and the minimum pressure is

$$p = 3{,}220 - 2{,}700 = +520 \text{ lbs. per sq. ft.}$$

The allowable pressure was 6,000 lbs. per sq. ft., so that the pressure is safe for a compact gravel. Where the walls were supported on the

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mushy clay it was necessary to extend the projection of the footing on the front side and to bring the resultant F to the center of the wall.

4. *Upward Pressure on Front Projection of Foundation.*—Where projections are used on the foundations of retaining walls it may be necessary to reinforce the base to prevent the projection breaking off in line with the face of the wall. The bending moment of the upward pressure about the front face of the wall from Fig. 184 is

$$M = \tfrac{1}{2}(5{,}920 + 4{,}120) \times 4 \times 2.1 \times 12$$
$$= 506{,}000 \text{ in.-lbs.}$$

The tension on the concrete at the bottom of the footing will be

$$f = M \cdot c/I = M \cdot d/2I = (506{,}000 \times 27)/157{,}464$$
$$= 88 \text{ lbs. per sq. in.}$$

Since the ultimate strength of the concrete in tension is approximately 200 lbs. per sq. in., no reinforcing is required. However, $\frac{3}{4}''$ □ bars were placed 18″ centers and 3″ from the bottom of the foundation.

Data.—The coefficients of friction and other data for various materials are given in Table XX. The conditions of surface and amount of moisture cause wide variations in the coefficients.

TABLE XX a.

COEFFICIENTS OF FRICTION.

Materials.	Coefficients.	Materials.	Coefficients.
Dry masonry on dry masonry	0.6 to 0.7	Masonry on dry clay	0.5 to 0.6
Masonry on masonry with wet mortar	0.75	Masonry on moist clay	0.33
Timber on stone	0.4	Earth on earth	0.25 to 1.0
Iron on stone	0.3 to 0.7	Hard brick on hard brick	0.7
Timber on timber	0.2 to 0.5	Concrete blocks on concrete blocks	0.65

TABLE XX b.

ANGLES OF REPOSE, ϕ, FOR MATERIALS.

Materials.	ϕ	Materials.	ϕ
Earth, loam	30° to 45°	Clay	25° to 45°
Sand, dry	25° to 35°	Gravel	30° to 40°
Sand, moist	30° to 45°	Cinders	25° to 40°
Sand, wet	15° to 30°	Coke	30° to 45°

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TABLE XX c.

Allowable Pressure on Foundations.

Material.	Pressure in Tons per Sq. Ft.
Soft clay	1 to 2
Ordinary clay and dry sand mixed with clay	2 to 3
Dry sand and clay	3 to 4
Hard clay and firm, coarse sand	4 to 6
Firm, coarse sand and gravel	6 to 8
Bed rock	15 and up.

TABLE XX d.

Allowable Pressure on Masonry.

Materials.	Pressure in Tons per Sq. Ft.
Common brick, Portland cement mortar	12
Paving brick, Portland cement mortar	15
Rubble masonry, Portland cement mortar	12
Sandstone, first class masonry	20
Limestone, " " "	25
Granite, " " "	30
Portland cement concrete, 1-2-4	25
" " " 1-3-6	20

TABLE XX e.

Weight, Specific Gravity and Crushing Strength of Masonry

Materials.	Weight in Pounds per Cubic Foot.	Specific Gravity.	Crushing Strength in Pounds per Square Inch
Sandstone	150	2.4	4,000 to 15,000
Limestone	160	2.6	6,000 to 20,000
Trap	180	2.9	19,000 to 33,000
Marble	165	2.7	8,000 to 20,000
Granite	165	2.7	8,000 to 20,000
Paving brick, Portland cement	150	2.4	2,000 to 6,000
Stone concrete, Portland cement	140 to 150	2.2 to 2.4	2,500 to 4,000
Cinder concrete, Portland cement	112	1.8	1,000 to 2,500

TABLE XX f.

Weight of Different Materials.

Materials.	Wt. per Cu. Ft. Lbs.	Materials.	Wt. per Cu. Ft. Lbs.
Loam, loose	75 to 90	Sand, wet	110 to 120
Loam, rammed	90 to 100	Gravel	120 to 135
Sand, dry	90 to 110	Soft flowing mud	105 to 120

For specifications for concrete, plain and reinforced, see Appendix III.

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For the design of reinforced concrete retaining walls, specifications for retaining walls and other data, see the author's "The Design of Walls, Bins and Grain Elevators," second edition.

STRESSES IN BIN WALLS.—The problem of the calculation of pressures on bin walls is similar to the problem of the calculation of pressures on retaining walls; but in the case of bin walls the material is limited in extent and the condition of static equilibrium is disturbed by drawing the material from the bottom of the bin. For plane bin walls where the plane of rupture cuts the free surface of the material (shallow bins), the formulas developed for retaining walls are directly applicable if friction on the wall is considered. The graphic solution will be found the simplest and most direct for any particular case. The following analyses of the calculations of stresses in bins have been abstracted from the author's "The Design of Walls, Bins and Grain Elevators," second edition.

STRESSES IN SHALLOW BINS.—The problem of the calculation of the pressures on bin walls is the same as the problem of the calculation of pressures on retaining walls. The forces acting on bin walls depend upon the weight, angle of repose, moisture, etc., of the material, which are variable factors, but are less variable than for the filling of retaining walls.

Algebraic Solution.—The same nomenclature will be used as in retaining walls except that P' will be used to indicate the pressure obtained by means of Cain's formulas when $z = \phi'$, N' will indicate the normal component of P', and N will indicate the normal pressure on the wall when $\phi' = 0$. This analysis applies to shallow bins, only.*

Case 1. Vertical Wall, Surface Level. Angle $z = \phi'$.

$$P' = \tfrac{1}{2} w \cdot h^2 \frac{\cos^2 \phi}{\cos \phi' \left(1 + \sqrt{\dfrac{\sin(\phi + \phi') \sin \phi}{\cos \phi'}}\right)^2} \qquad (101)$$

$$N' = P' \cdot \cos \phi' \qquad (102)$$

If

$$\phi' = \phi$$

$$P' = \tfrac{1}{2} w \cdot h^2 \frac{\cos \phi}{(1 + \sin \phi \sqrt{2})^2} \qquad (103)$$

$$N' = P' \cdot \cos \phi \qquad (102')$$

* A shallow bin is one where the plane of rupture cuts the free surface of the filling.

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If $\phi' = 0$, which corresponds to a smooth wall,

$$N = \tfrac{1}{2}w \cdot h^2 \cdot \tan^2\left(45^\circ - \frac{\phi}{2}\right) \tag{105}$$

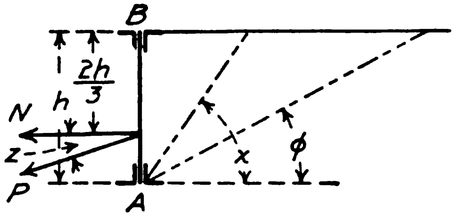

FIG. 185.

TABLE XXI.

CONSTANTS FOR STEEL PLATE BINS, CASE 1.

Material.	ϕ Degrees.	ϕ' Degrees.	W Lbs. Per Cu. Ft.	P' Lbs.	N' Lbs.	N Lbs.
Bituminous coal..........	35	18	50	$6.13h^2$	$5.83h^2$	$6.75h^2$
Anthracite coal...........	27	16	52	$8.73h^2$	$8.39h^2$	$9.77h^2$
Sand	34	18	90	$11.50h^2$	$10.93h^2$	$12.72h^2$
Ashes.......................	40	31	40	$4.02h^2$	$3.44h^2$	$4.34h^2$

Case 2. Vertical Wall, Surface Surcharged at Angle δ. Angle $z = \phi'$.

$$P' = \tfrac{1}{2}w \cdot h^2 \frac{\cos^2\phi}{\cos\phi'\left(1 + \sqrt{\frac{\sin(\phi+\phi')\sin(\phi-\delta)}{\cos\phi' \cdot \cos\delta}}\right)^2} \tag{106}$$

$$N' = P' \cdot \cos\phi' \tag{106'}$$

If

$$\delta = \phi$$

$$P' = \tfrac{1}{2}w \cdot h^2 \frac{\cos^2\phi}{\cos\phi'} \tag{107}$$

$$N' = P' \cdot \cos\phi' = \tfrac{1}{2}w \cdot h^2 \cdot \cos^2\phi \tag{108}$$

If

$$\phi' = 0$$

$$N = \tfrac{1}{2}w \cdot h^2 \cdot \cos^2\phi \tag{109}$$

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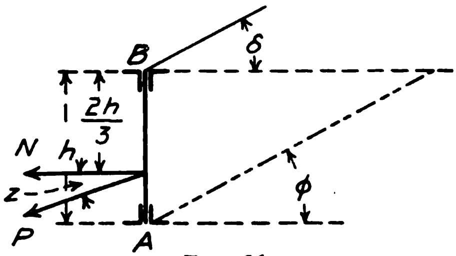

FIG. 186.

TABLE XXII.

CONSTANTS FOR STEEL PLATE BINS, CASE 2. $\delta = \phi$.

Material.	ϕ Degrees.	ϕ' Degrees.	W Lbs. Per Cu. Ft.	P' Lbs.	N' Lbs	N Lbs.
Bituminous coal........	35	18	50	$17\ 65h^2$	$16.75h^2$	$16.75h^2$
Anthracite coal	27	16	52	$21\ 45h^2$	$20.50h^2$	$20.50h^2$
Sand	34	18	90	$32.50h^2$	$30\ 90h^2$	$30.90h^2$
Ashes..........................	40	31	40	$13.70h^2$	$11.73h^2$	$11.73h^2$

Case 3. Vertical Wall, Surcharge Negative $=\delta$. Angle $z = \phi'$.

$$P' = \tfrac{1}{2} w \cdot h^2 \frac{\cos^2 \phi}{\cos \phi' \left(1 + \sqrt{\dfrac{\sin (\phi + \phi') \sin (\phi + \delta)}{\cos \phi' \cdot \cos \delta}} \right)^2} \qquad (110)$$

$$N' = P' \cdot \cos \phi' \qquad (111)$$

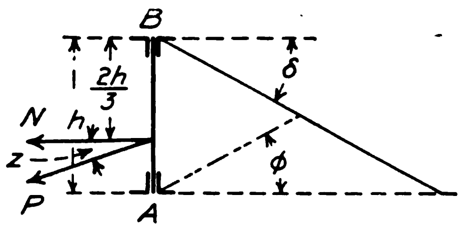

FIG. 187.

If

$$\phi' = 0$$

$$N = \tfrac{1}{2} w \cdot h^2 \frac{\cos^2 \phi}{\left(1 + \sqrt{\dfrac{\sin \phi \sin (\phi + \delta)}{\cos \delta}} \right)^2} \qquad (112)$$

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TABLE XXIII.

CONSTANTS FOR STEEL PLATE BINS, CASE 3. $\delta = -\phi$.

Material.	ϕ Degrees.	ϕ' Degrees.	W Lbs. Per Cu. Ft.	P' Lbs.	N' Lbs.	N Lbs.
Bituminous coal..........	35	18	50	$4.49h^2$	$4.27h^2$	$5.13h^2$
Anthracite coal...	27	16	52	$6.64h^2$	$6.38h^2$	$7.64h^2$
Sand	34	18	90	$8.44h^2$	$8.00h^2$	$9.61h^2$
Ashes...........	40	31	40	$2.85h^2$	$2.45h^2$	$3.23h^2$

Case 4. Wall Sloping Outward. $\theta < 90° + \phi'$. *Surface Level.*

$$P' = \tfrac{1}{2}w \cdot h^2 \frac{\sin^2(\theta - \phi)}{\sin(\phi' + \theta)\sin^2\theta\left(1 + \sqrt{\dfrac{\sin(\phi + \phi')\sin\phi}{\sin(\phi' + \theta)\sin\theta}}\right)^2} \tag{113}$$

$$N' = P' \cdot \cos\phi' \tag{114}$$

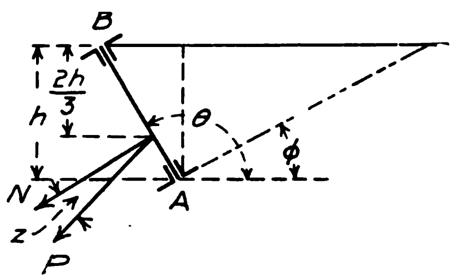

FIG. 188.

Case 5. Wall Sloping Outward. $\theta < 90° + \phi'$. *Surface Surcharged.*

$$P' = \tfrac{1}{2}w \cdot h^2 \frac{\sin^2(\theta - \phi)}{\sin(\phi' + \theta)\sin^2\theta\left(1 + \sqrt{\dfrac{\sin(\phi + \phi')\sin(\phi - \delta)}{\sin(\phi' + \theta)\sin(\theta - \delta)}}\right)^2} \tag{115}$$

$$N' = P' \cdot \cos\phi' \tag{116}$$

Case 6. Wall Sloping Outward. $\theta > 90° + \phi'$. *Surface Level.*

$$P = \tfrac{1}{2}w \cdot h^2 \cdot \tan^2\left(45° - \frac{\phi}{2}\right)$$

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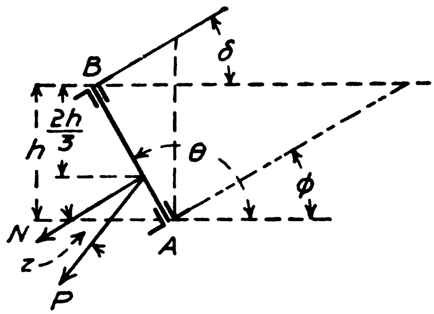

FIG. 189.

$$W = weight\ \triangle ABC = \frac{w \cdot \tan\theta \cdot h^2}{2}$$

$$E = \sqrt{W^2 + P^2}$$

$$= \tfrac{1}{2} w \cdot h^2 \sqrt{\tan^2\theta + \tan^4\left(45^\circ - \frac{\phi}{2}\right)} \qquad (117)$$

$$\tan(\theta + z - 90^\circ) = \frac{\tan\theta}{\tan^2\left(45^\circ - \frac{\phi}{2}\right)} \qquad (118)$$

$$Q = E \cdot \cos z$$

$$T = E \cdot \sin z$$

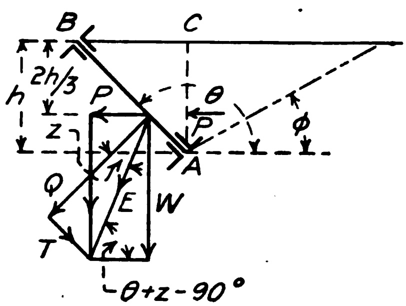

FIG. 190.

STRESSES IN SHALLOW BINS, Graphic Solution.—The graphic solution will be given for two cases which frequently occur in practice.

Graphic Solution. Hopper Bin, Level Full.—The calculation of stresses in bins by means of graphics will be illustrated by the follow-

ing problem taken from "The Design of Walls, Bins and Grain Elevators." The bin, section of which is shown in Fig. 191, is filled with coal weighing 58 lbs. per cu. ft., and having an angle of repose $\phi = 30°$. The total pressure on the plane A–H is

$$P_1 = \tfrac{1}{2} w \cdot h^2 \frac{1 - \sin\phi}{1 + \sin\phi} = 3{,}130 \text{ lbs.}$$

acting horizontally through a point 12 ft. below the top surface. Now, to find the pressure P_2 on the plane G–A, produce P_1 until it intersects

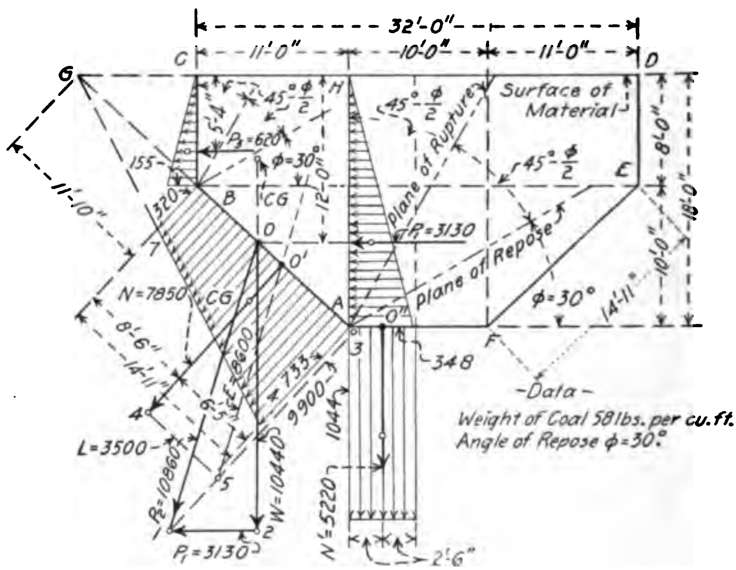

FIG 191.

the line O–2 = the weight of triangle AHG = 10,440 lbs. at O, and by constructing O–1 = P_2 = 10,860 lbs. P_2 is parallel to E in Fig. 191. The normal pressure on A–g is 9,900 lbs. Now A–1 = 9,900 lbs. acts through the center of gravity of triangle $AG4$, and is equal to the area of $AG4 \times w$. The normal unit pressure at A is 733 lbs. per sq. ft., and the normal unit pressure at B is 320 lbs. per sq. ft. The normal pressure on AB acts through the center of gravity of the shaded area, and is $N = 7{,}850$ lbs. Also by construction $E = 8{,}600$ lbs. The pres-

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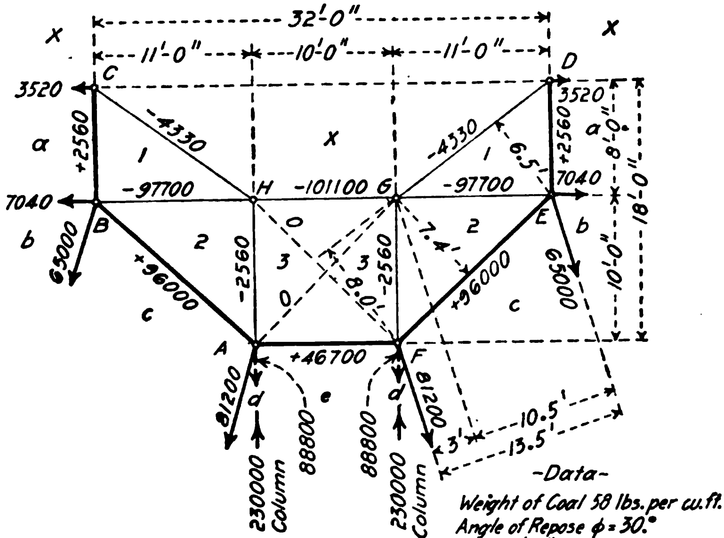

FIG. 192

Algebraic Moments.

Center of Moments, E.

Stress GD.

$-GD \times 6.5' - 3520 \times 8' = 0$

$GD = -4330$

Stress FG.

$-FG \times 11' - 3520 \times 8' = 0$

$FG = -2560$

Center of Moments, F.

Stress GH.

$-GH \times 10' - 3520 \times 18' - 7040 \times 10' - 65000 \times 13.5' = 0$

$GH = -101100$

Stress GE.

$-GE \times 10' + 4330 \times 8' - 3520 \times 18' - 7040 \times 10'$
$- 65000 \times 13.5 = 0 \quad GE = -97700$

Center of Moments, G.

Stress ED.

$-ED \times 11' + 3520 \times 8' = 0$

$ED = +2560$

Stress FE.

$FE \times 7.4' - 3520 \times 8' - 65000 \times 10.5 = 0$

$FE = +96000$

Stress AF.

$AF \times 10' + 81200 \times 3' - 65000 \times 10.5'$
$-3520 \times 8' = 0. \quad AF = +46700$

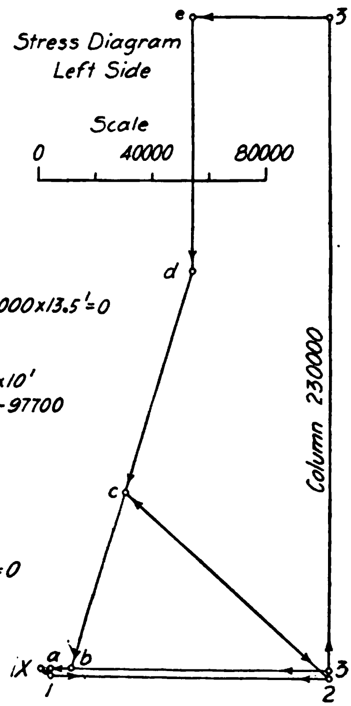

FIG. 193.

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sure on bottom A–F is equal to $18 \times 58 = 1,044$ lbs. per sq. ft. The pressure on the wall C–B is

$$P_3 = \tfrac{1}{2} w \cdot h^2 \frac{1 - \sin\phi}{1 + \sin\phi} = 620 \text{ lbs.}$$

Calculation of Stresses in Framework.—The loads on the bin walls are carried by a transverse framework as shown in Fig. 192, spaced 17′ 0″ center to center The loads at the joints act parallel to the pressures as previously calculated, and the loads can be calculated in the same manner as for a simple beam loaded with a similar loading. The stresses are calculated by graphic resolution and by algebraic moments as shown in Fig. 192 and Fig. 193.

Hopper Bin, Top Surface Heaped.—The bin in Fig. 194 is heaped at the angle of repose, $\phi = 30°$. To calculate the pressure on side

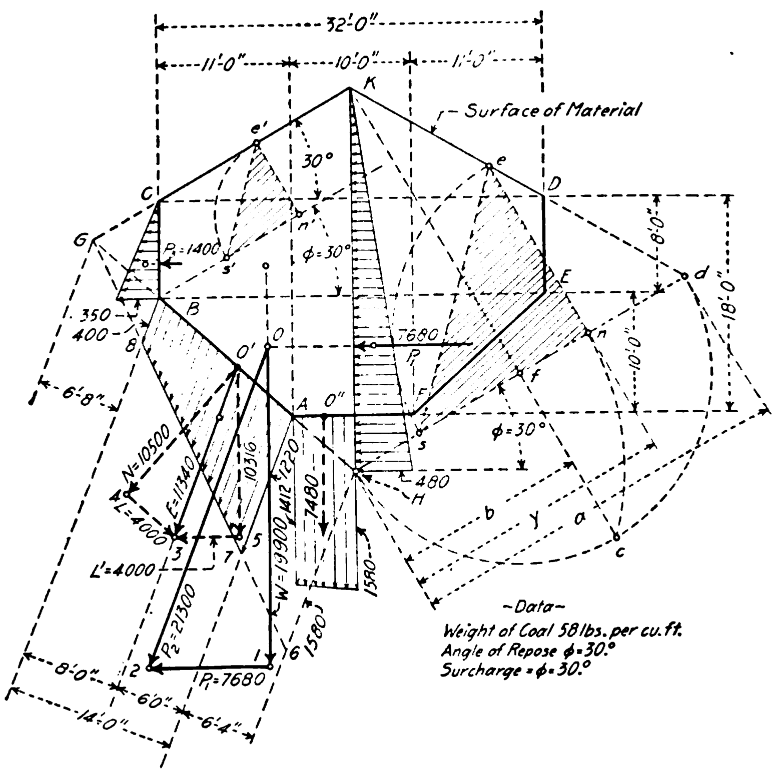

FIG. 194.

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A–B, proceed as follows: Locate points G and H, and calculate the horizontal pressure $P_1 = 7{,}680$ lbs., acting on the plane H–K at $\frac{1}{3}HK$ above H. Pressure P_1 was calculated by the graphic method described in Fig. 179. Produce P_1 until it intersects at O the line of action of the weight of the triangle GHK acting through the center of gravity of the triangle. From O lay off O–1 $= W' = 19{,}900$ lbs., acting downwards, and from 1 lay off 1–2 $= P_1 = 7{,}680$ lbs., acting to the left. Then O–2 $= P_2 = 21{,}300$ lbs. Now $P_2 =$ area triangle $6GH \cdot w$, and $E =$ area 8–B–A–$5 \cdot w = 11{,}340$ lbs. Force E acts through the center of gravity of area 8–B–A–7. The horizontal pressure on plane C–B $= 1{,}400$ lbs. $=$ area $s'c'n' \cdot w$. The vertical pressure on the left-hand side of the bottom A–F is 7,480 lbs., acting through the center of gravity of the pressure polygon. The vertical unit pressure at A is 1,412 lbs. per sq. ft.

STRESSES IN DEEP BINS.—Where the plane of rupture cuts the sides of the bin the solution for shallow bins does not apply.

Nomenclature.—The following nomenclature will be used:

$\phi =$ the angle of repose of the filling;

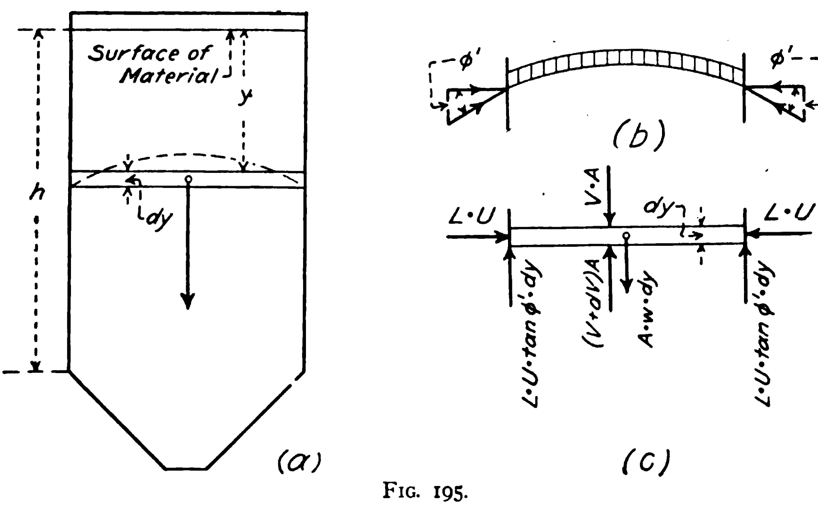

FIG. 195.

$\phi' =$ the angle of friction of the filling on the bin walls;

$\mu = \tan \phi =$ coefficient of friction of filling on filling;

$\mu' = \tan \phi' =$ coefficient of friction of filling on the bin walls;

$x =$ angle of rupture;

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w = weight of filling in lbs. per cu. ft.;
V = vertical pressure of the filling in lbs. per sq. ft.;
L = lateral pressure of the filling in lbs. per sq. ft.;
A = area of bin in sq. ft.;
U = circumference of bin in feet;
$R = A/U$ = hydraulic radius of bin.

Solution.—The bin in (a) Fig. 195, has a uniform area A, a constant circumference U, and is filled with a granular material weighing w per unit of volume, and having an angle of repose ϕ. Let V be the vertical pressure, and L be the lateral pressure at any point, both V and L being assumed as constant for all points on the horizontal plane. (More correctly V and L will be constant on the surface of a dome as in (b).)

The weight of the granular material between the sections of y and $y+dy = A\cdot w\cdot dy$; the total frictional force acting upwards at the circumference will be $= L\cdot U\cdot \tan\phi'\cdot dy$; the total perpendicular pressure on the upper surface will be $= V\cdot A$; and the total pressure on the lower surface will be $= (V+dV)A$.

Now these vertical pressures are in equilibrium, and

$$V\cdot A-(V+dV)A+A\cdot w\cdot dy-L\cdot U\cdot \tan\phi'\cdot dy=0$$

and

$$dV=\left(w-L\cdot\tan\phi'\frac{U}{A}\right)dy \qquad (119)$$

Now in a granular mass, the lateral pressure at any point is equal to the vertical pressure times k, a constant for the particular granular material, and

$$L=k\cdot V$$

Also let $\dfrac{A}{U}=R$ (the hydraulic radius), and $\tan\phi'=\mu'$.

Substituting the above in (119) we have

$$dV=\left(w-\frac{k\cdot V}{R}\mu'\right)dy$$

Now let

$$\frac{k\cdot\mu'}{R}=n \qquad (120)$$

and

$$\frac{dV}{w-n\cdot V}=dy \qquad (121)$$

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Integrating (121) we have

$$\log (w - n \cdot V) = - n \cdot y + C \tag{122}$$

Now if $y = 0$, then $V = 0$, and $C = \log w$, and (122) reduces to

$$\log \left(\frac{w - n \cdot V}{w} \right) = - n \cdot y$$

and

$$\frac{w - n \cdot V}{w} = \frac{1}{e^{n \cdot y}} = e^{-n \cdot y}$$

where e is the base of the Naperian system of logarithms. Solving for V we have

$$V = \frac{w}{n} (1 - e^{-n \cdot y}) \tag{123}$$

Substituting the value of n from (120), we have

$$V = \frac{w \cdot R}{k \cdot \mu'} (1 - e^{-\frac{k \cdot \mu' \cdot y}{R}}) \tag{124}$$

Now if h be taken as the depth of the granular material at any point, we will have

$$V = \frac{w \cdot R}{k \cdot \mu'} (1 - e^{-\frac{k \cdot \mu' \cdot h}{R}}) \tag{125}$$

Also since

$$L = k \cdot V$$

$$L = \frac{w \cdot R}{\mu'} (1 - e^{-\frac{k \cdot \mu' \cdot h}{R}}) \tag{126}$$

Now if w is taken in lbs. per cu. ft., and R in feet, the pressure will be given in lbs. per square foot.

Naperian or hyperbolic logarithms of numbers from 1 to 100 are given in Table XXIV.

Now both μ' and k can only be determined by experiment on the particular granular material and kind of bin. For wheat and a wooden bin k varies from 0.4 to 0.6.

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TABLE XXIV.

Hyperbolic or Naperian Logarithms.

N.	Log.	N.	Log.	N.	Log.
1.00	0.0000	3.65	1.2947	6.60	1.8871
1.05	0.0488	3.70	1.3083	6.70	1.9021
1.10	0.0953	3.75	1.3218	6.80	1.9169
1.15	0.1398	3.80	1.3350	6.90	1.9315
1.20	0.1823	3.85	1.3481	7.00	1.9459
1.25	0.2231	3.90	1.3610	7.20	1.9741
1.30	0.2624	3.95	1.3737	7.40	2.0015
1.35	0.3001	4.00	1.3863	7.60	2.0281
1.40	0.3365	4.05	1.3987	7.80	2.0541
1.45	0.3716	4.10	1.4110	8.00	2.0794
1.50	0.4055	4.15	1.4231	8.25	2.1102
1.55	0.4383	4.20	1.4351	8.50	2.1401
1.60	0.4700	4.25	1.4469	8.75	2.1691
1.65	0.5008	4.30	1.4586	9.00	2.1972
1.70	0.5306	4.35	1.4701	9.25	2.2246
1.75	0.5596	4.40	1 4816	9.50	2.2513
1.80	0.5878	4.45	1.4929	9.75	2.2773
1.85	0.6152	4.50	1.5041	10.00	2.3026
1.90	0.6419	4.55	1.5151	11.00	2.3979
1.95	0.6678	4.60	1.5261	12.00	2.4849
2.00	0.6931	4.65	1.5369	13.00	2.5649
2.05	0.7178	4.70	1.5476	14.00	2.6391
2.10	0.7419	4.75	1.5581	15.00	2.7081
2.15	0.7655	4.80	1.5686	16.00	2.7726
2.20	0.7885	4.85	1.5790	17.00	2.8332
2.25	0.8109	4.90	1.5892	18.00	2.8904
2.30	0.8329	4.95	1 5994	19.00	2.9444
2.35	0.8544	5.00	1.6094	20.00	2.9957
2.40	0.8755	5.05	1.6194	21.00	3.0445
2.45	0.8961	5.10	1.6292	22.00	3.0910
2.50	0.9163	5.15	1.6390	23.00	3.1355
2.55	c.9361	5.20	1.6487	24.00	3.1781
2.60	0.9555	5.25	1.6582	25.00	3.2189
2.65	0.9746	5.30	1.6677	26.00	3.2581
2.70	0.9933	5.35	1.6771	27.00	3.2958
2.75	1.0116	5.40	1.6864	28.00	3.3322
2.80	1.0296	5.45	1.6956	29.00	3.3673
2.85	1.0473	5.50	1.7047	30.00	3.4012
2.90	1.0647	5.55	1.7138	31.00	3.4340
2.95	1.0818	5.60	1.7228	32.00	3.4657
3.00	1.0986	5.65	1.7317	33.00	3.4965
3.05	1.1154	5.70	1.7405	34.00	3.5264
3.10	1.1314	5.75	1.7492	35.00	3.5553
3.15	1.1474	5.80	1.7579	40.00	3.6889
3.20	1.1632	5.85	1.7664	45.00	3.8066
3.25	1.1787	5.90	1.7750	50.00	3.9120
3.30	1.1939	5.95	1.7834	60.00	4.0943
3.35	1.2090	6.00	1.7918	70.00	4.2485
3.40	1.2238	6.10	1.8083	80.00	4.3820
3.45	1.2384	6.20	1.8245	90.00	4.4998
3.50	1.2528	6.30	1.8405	100.00	4.6052
3.55	1.2669	6.40	1.8563		
3.60	1.2809	6.50	1.8718		

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An approximate value of k may be calculated from the formula

$$k = \frac{1 - \sin \phi}{1 + \sin \phi}$$

where ϕ = the angle of internal friction of the material. For a deep bin the maximum pressure as given by formula (126) is independent of k, and its exact determination is therefore not important.

For angles of internal friction, see Table XXV, and for angles of friction on bin walls, see Table XXVI.

For methods of calculating the stresses in hopper bins with the top surface surcharged, in suspension bunkers, and the calculation of the stresses in bin bottoms and circular girders, see the author's "The Design of Walls, Bins and Grain Elevators."

Angle of Repose.—The angle of repose and the weight of different materials are given in Table XXV.

TABLE XXV.

WEIGHT AND ANGLE OF REPOSE OF COAL, COKE, ASHES AND ORE.

Material.	Weight Lbs. per Cu. Ft.	Angle of Repose ϕ in Degrees.	Authority.
Bituminous coal.	50	35	Link Belt Machinery Co.
Bituminous coal.	47	35	Link Belt Engineering Co.
Bituminous coal.	47 to 56		Cambria Steel.
Anthracite coal.	52	27	Link Belt Machinery Co.
Anthracite coal.	52.1	27	Link Belt Engineering Co.
Anthracite coal fine.		27	K. A. Muellenhoff.
Anthracite coal	52 to 56		Cambria Steel.
Slaked coal.		45	Wellman-Seaver-Morgan Co.
Slaked coal.	53	37½	Gilbert and Barth.
Coke.	23 to 32		Cambria Steel.
Ashes.	40	40	Link Belt Machinery Co.
Ashes, soft coal.	40 to 45		Cambria Steel.
Ore, soft iron.		35	Wellman-Seaver-Morgan Co.

Coal, ore, etc., will give an angle of $\phi = 40°$ if the material is dry, but if the material is wet the angle of repose may be increased to nearly 90°.

Angle of Friction on Bin Walls.—The values in Table XXVI may be used in the absence of more accurate data.

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TABLE XXVI.

ANGLE OF FRICTION OF DIFFERENT MATERIALS ON BIN WALLS.

Material.	Steel Plate. ϕ' in Degrees.	Wood Cribbed. ϕ' in Degrees.	Concrete. ϕ' in Degrees.
Bituminous coal.	18	35	35
Anthracite coal.	16	25	27
Ashes.	31	40	40
Coke.	25	40	40
Sand.	18	30	30

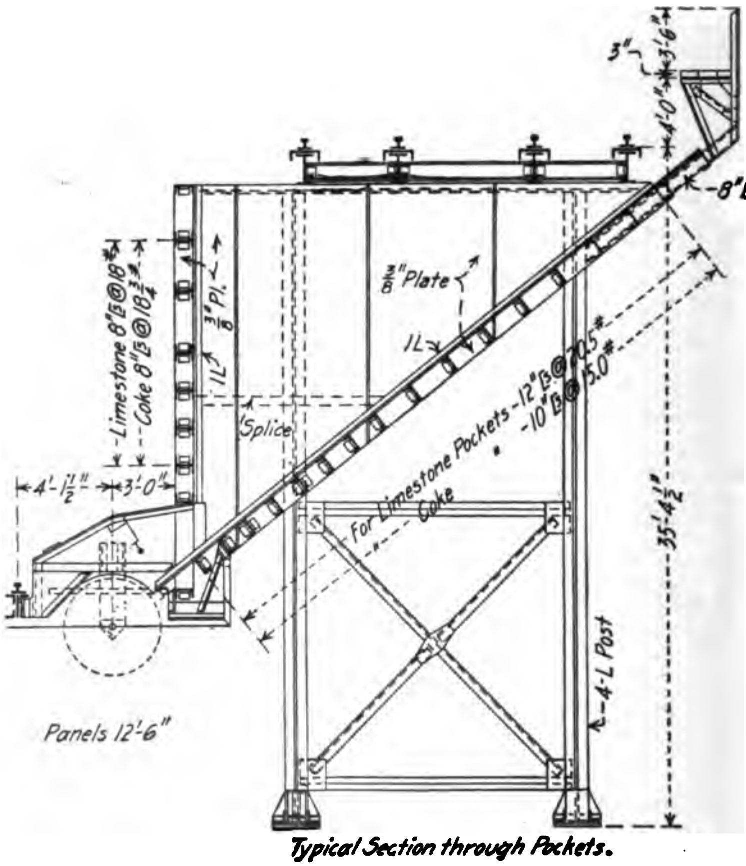

FIG. 196. COKE AND STONE BINS, LACKAWANNA STEEL CO.

Self-cleaning Hoppers.—In order to have hoppers self-cleaning when the material is moist it is necessary to have the hopper bottoms

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slope at an angle considerably in excess of the angle of repose ϕ or angle of friction ϕ'.

Ore pockets on the Great Lakes are made with hopper bottoms at an angle of 48° 40′ to 50° 45′, but the majority are at an angle of 49° 45′. Bituminous coal will slide down a steel chute at an angle of 40°

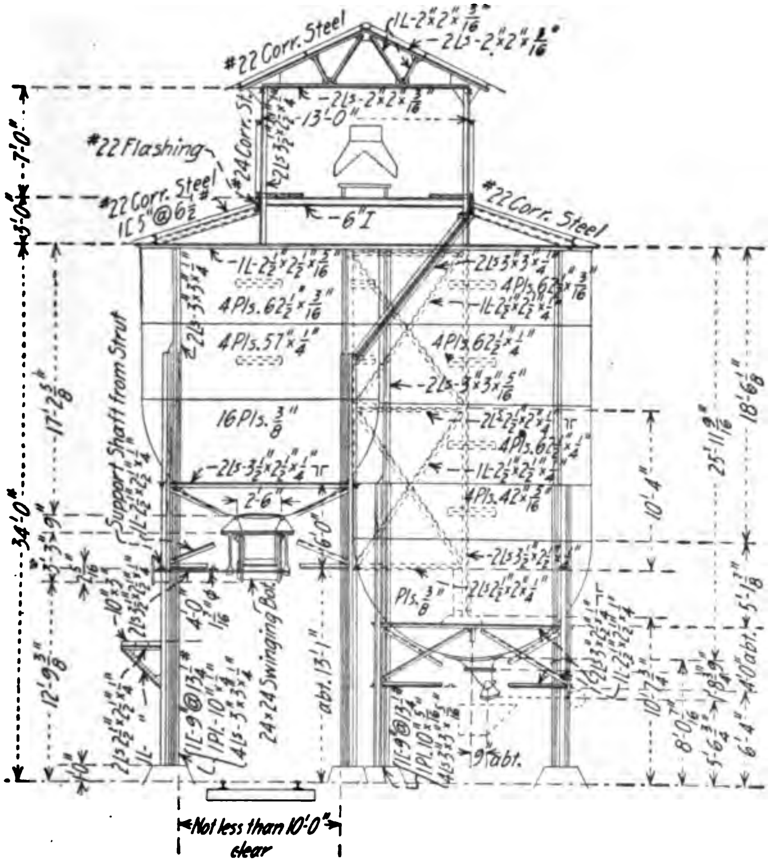

FIG. 197 ELEVATION CIRCULAR STEEL ORE BIN FOR OLD DOMINION COPPER MINING CO.

and a wooden chute at an angle of 45°. Anthracite coal will slide down a steel chute at an angle of 30° and down a wooden chute at an angle of 35°.

DESIGN OF BINS.—Bins are usually subjected to sudden loads and vibrations and should be designed with two thirds the allowable unit stresses for dead loads given in §§ 33 to 41, inclusive, in "Speci-

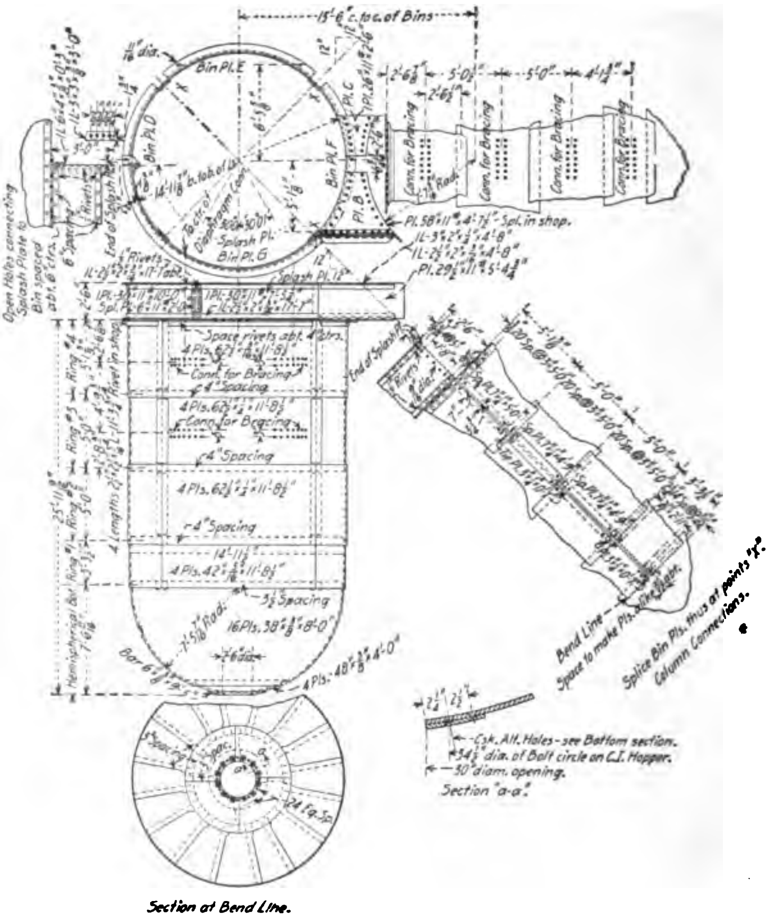

FIG. 198. DETAILS FOR CIRCULAR BINS FOR OLD DOMINION COPPER MINING CO.

fications for Steel Mine Structures," Appendix I. For details and the design of flat and buckle plates, see Chapter XIV.

Bins are made of timber, of structural steel, or of concrete, or the different materials may be used in combination.

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TYPES OF BINS.—The most common types are (1) the suspension bunker, (2) the hopper bin, and (3) the circular bin.

Suspension Bunkers.—Suspension bunkers are made by suspending a steel framework from two side members, the weight of the filling

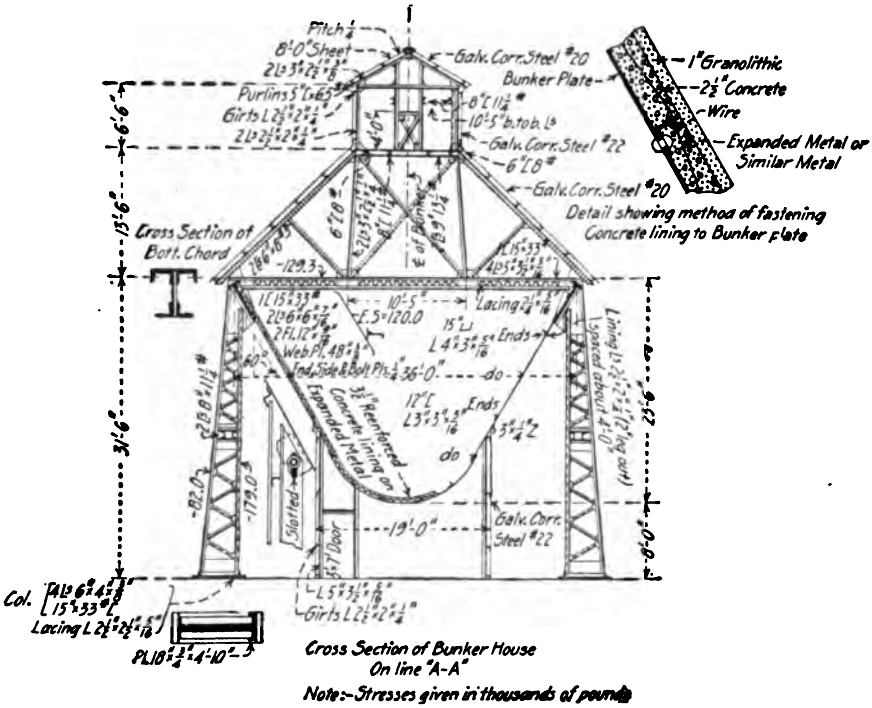

FIG. 199. COAL BUNKERS, RAPID TRANSIT SUBWAY, NEW YORK, N. Y.

causing the sides to assume the curve of an equilibrium polygon. The stresses in the plates of a true suspension bunker are pure tensile stresses. Steel suspension bunkers are commonly lined with a concrete lining about 1½ to 3½ in. thick, reinforced with wire fabric, to protect the metal of the bin.

Hopper Bins.—Hopper bins may be made of timber, steel, or reinforced concrete. A steel coke and stone bin, erected by the Lackawanna Steel Company is shown in Fig. 196. These bins were divided into panels 12 ft. 6 in. center to center, with double partitions at each panel point, leaving a clear length of 11 ft. 6 in. The bins are lined through-

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out with $\frac{3}{8}$ in. plates. All rivets in the floor are countersunk. The gates at the bottom of the bin are cylindrical and are revolved by a system of shafting and gears. There is an opening in the side of the drum, and when the drum is revolved this opening comes opposite the

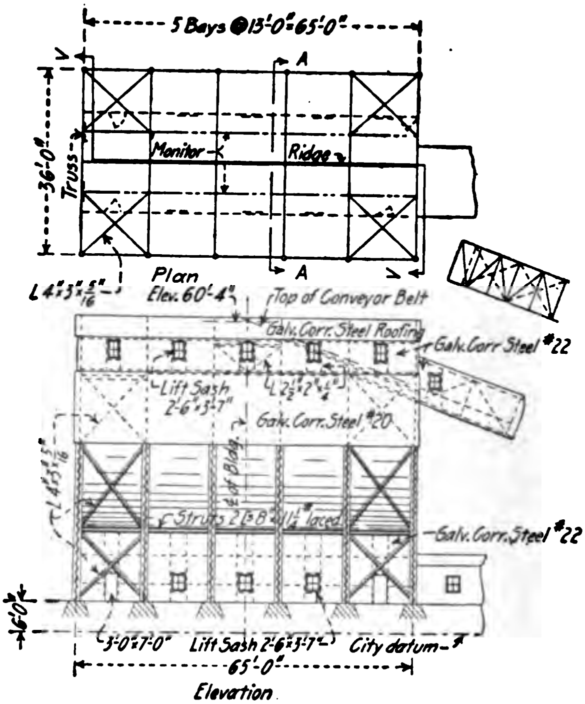

FIG. 200. COAL BUNKERS, RAPID TRANSIT SUBWAY, NEW YORK, N. Y.

opening in the bottom of the bin and the drum is filled. The drum is then revolved and the material is dumped into the larries.

Circular Bins.—Circular bins are made of both steel and reinforced concrete. A circular ore bin with a hemispherical bottom is shown in Fig. 197 and Fig. 198. Circular steel bins are used in the Quincy Rock House, as is shown in Fig. 109.

Bin Gates.—Many gates have been devised to make it possible to handle different materials quickly and easily. The gates in Fig. 201 are especially adapted to coal and similar materials. Gate (*b*) in Fig. 202 is a type of gate commonly used in ore bins.

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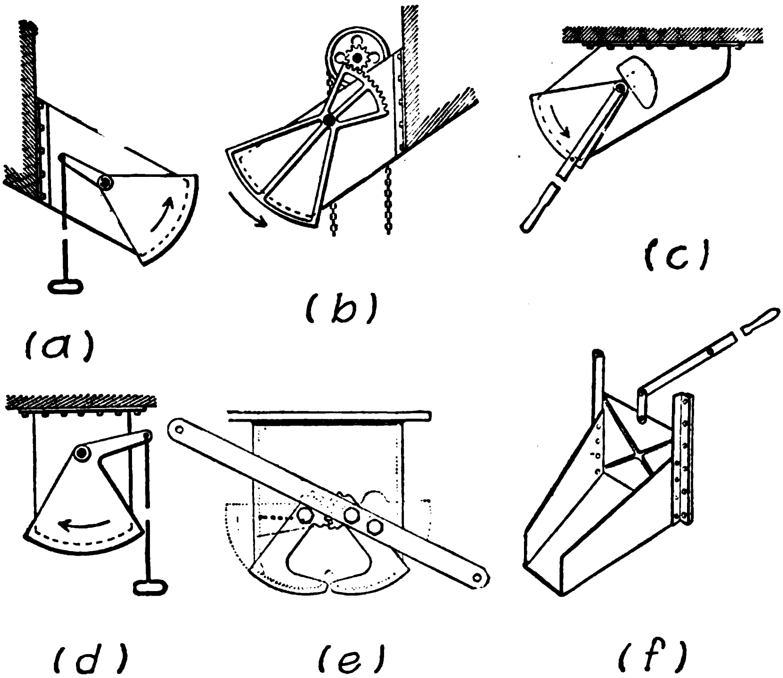

FIG. 201.

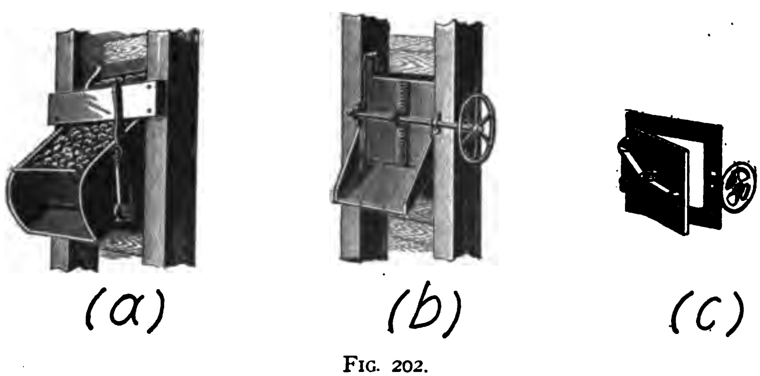

FIG. 202.

EXAMPLES OF BINS. Steel Coal Bin for Rapid Transit Subway.—A cross-section of a 1,000-ton suspension bunker built by the Rapid Transit Subway, New York City, is shown in Fig. 199 and Fig. 200. The bunker is supported on posts and is covered by corru-

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gated steel. The bin is lined with a layer of concrete 3½ in. thick, reinforced with expanded metal. The details of construction are plainly shown in the cuts.

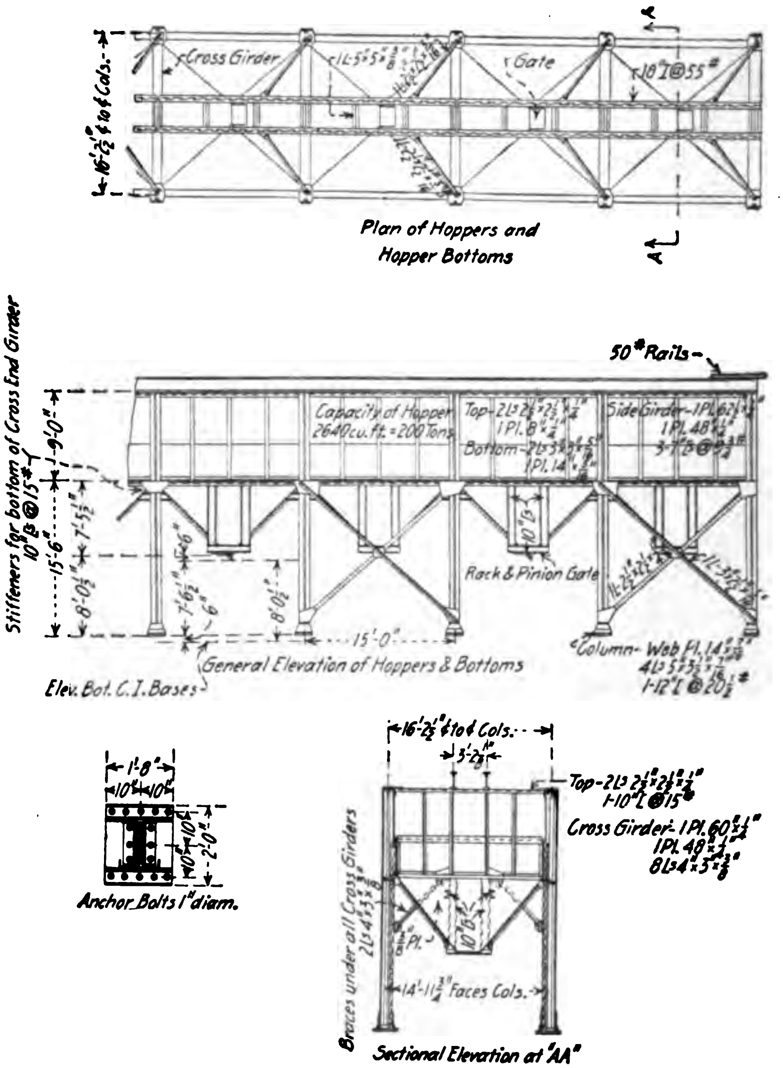

FIG. 203. HOPPER BIN CANANEA CONSOLIDATED COPPER CO., CANANEA, MEXICO.

Ore Bins for Cananea Consolidated Copper Company.—Detail drawings of a hopper ore bin built by the Cananea Consolidated Cop-

per Company are shown in Fig. 203. The ore is coarse and heavy and is dumped from cars on the top of the bins. The ore is drawn off through gates on the bottom and is carried away on a conveyor. The side plates are $\frac{1}{4}$ in. thick and are stiffened with channels spaced about 4 ft. apart The hopper plates are $\frac{3}{8}$ in. thick and are stiffened with 10 in. channels.

Steel Coal Bins for Davis Coal and Coke Co.—The steel coal bin shown in Fig. 204 was designed by the American Bridge Company for the Davis Coal and Coke Co. for the coke ovens at Coketon, W. Va.

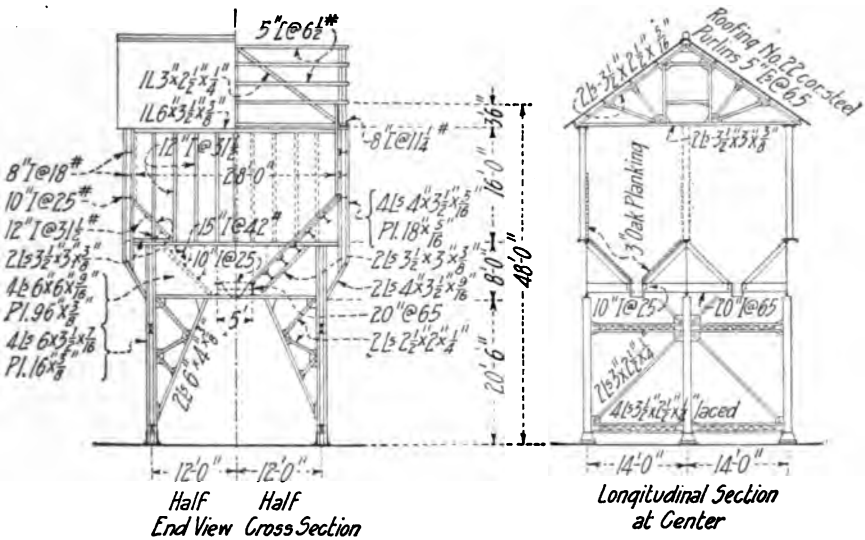

FIG. 204. STEEL COAL BINS AT COKETON, W. VA.

The framework is made of structural steel and is covered with corrugated steel. The bin is lined with 3 in. oak plank spiked to timber spiking pieces which are bolted to the steel beams. The bin is carried on plate girders each having a web plate 96 in. $\times \frac{3}{8}$ in., and top and bottom flanges of two angles $6'' \times 6'' \times \frac{9}{16}''$. The bin is filled by a belt conveyor passing over the top of the bin, as shown in Fig. 204. The coal is drawn from the bins through gates into cars and is hauled to the coke ovens. The capacity of the bin is 300 tons.

Ore House for Tonopah-Belmont Company.—The ore house at the Belmont shaft of the Tonopah-Belmont Development Company is

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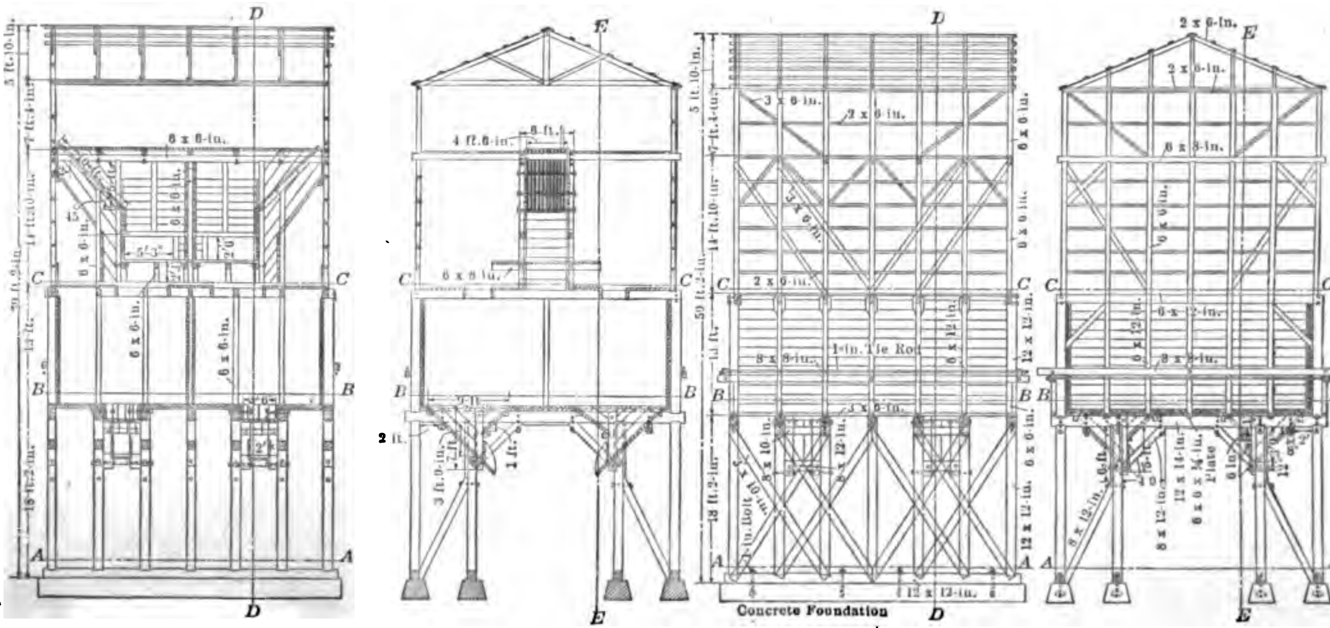

Section through E-E   Section through D-D   Side Elevation   End Elevation

FIG. 205. ORE HOUSE FOR TONOPAH-BELMONT COMPANY, TONOPAH, NEVADA.

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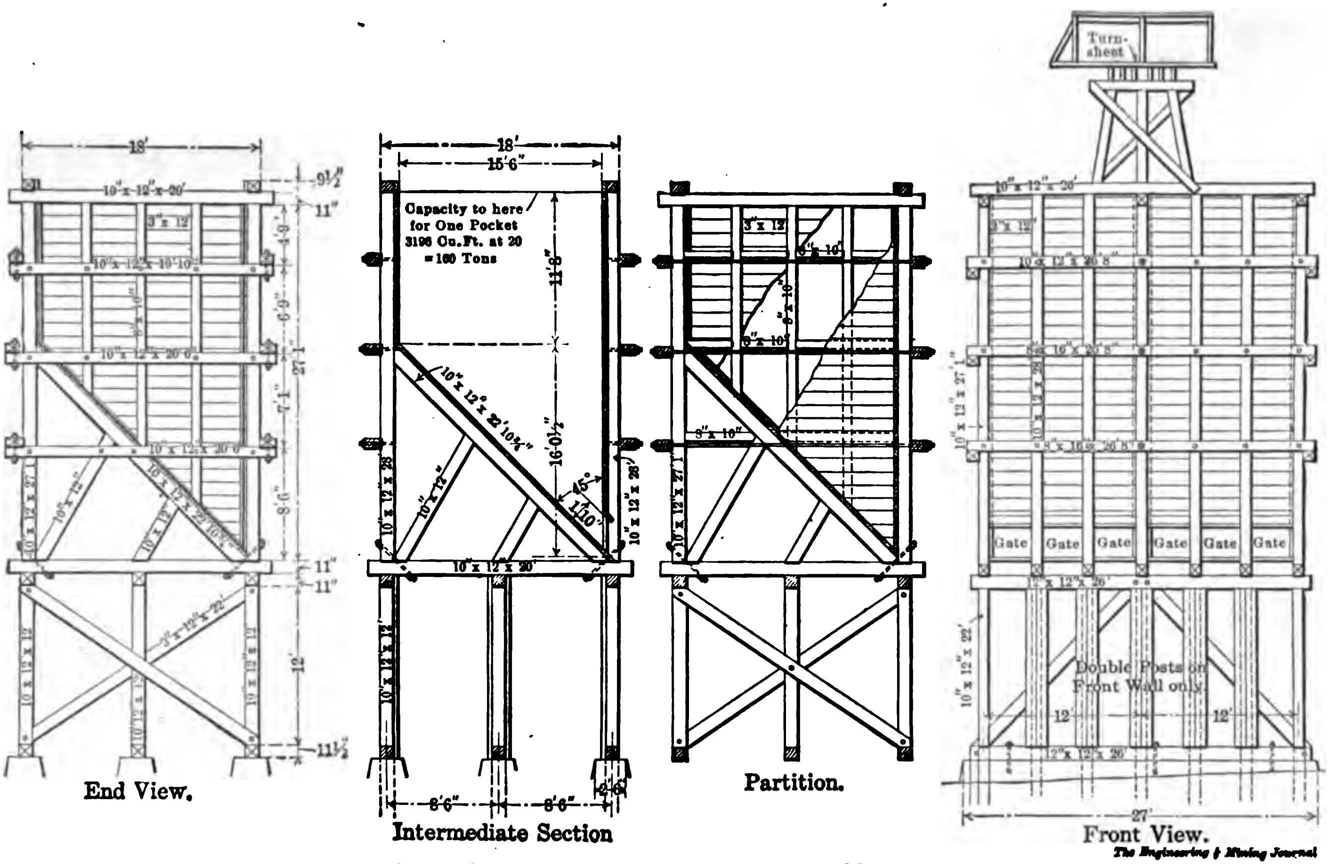

FIG. 206. TIMBER ORE BINS AT CANANEA, MEXICO.

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shown in Fig. 205. The bin has a capacity of approximately 750 tons and is divided into three compartments by a partition running longitudinally through the middle and transversely at one third the length. The bin for mill ore has a capacity of 525 tons and the bin for smelting ore 225 tons, making a total capacity of 750 tons. The ore bins are located 220 ft. from the collar of the shaft, from which the ore is carried in mine cars on the double track trestle. The ore is sorted on dry picking tables above the bin proper and is run over screens. The screens are made of cast manganese plates, 1 in. thick and have holes in rows 2 in. apart at the top and $2\frac{1}{4}$ in. at the bottom. These plates are set at an angle of 40 degrees and clean themselves well, the holes showing no tendency to bind. The oversize from the screens goes to the picking tables, where the waste is removed and is thrown into mine cars and trammed to the dump, while the sorted mill ore is scraped off into hoppers and is taken to the bin compartments.

The main timbers of the ore house are $12'' \times 12''$ with $3'' \times 10''$ and $3'' \times 6''$ bracing. The bins were lined with $6'' \times 12''$ timber lining. The ore bin was built of Oregon spruce, which costs $37.50 per M, and was erected by carpenters who were paid from $5.00 to $5.50 per eight-hour shift.

Cananea Ore Bins.—Timber ore bins designed for use at the mines at Cananea, Mexico, are shown in Fig. 206. The bottoms of the bins make an angle of 45 degrees with the horizontal, and are covered with steel plate $\frac{3}{16}$ in. thick. The details are shown in the cut and need no explanation.

For examples of reinforced concrete bins, and for additional examples of steel bins, see the author's "The Design of Walls, Bins and Grain Elevators."

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CHAPTER XI.

The Design of Coal Washers.

Introduction.—Coal as found in the mine consists of coal proper associated in greater or less quantities with slate, sulphur, usually in the form of iron pyrites, ash, commonly called "bone" and in many mines with clay or dirt. If the impurities occur in thick layers or masses they are readily removed by hand picking in a breaker, but where they are intimately dispersed through the coal the impurities can only be removed by what is known as washing.

The object of washing coal is to remove the impurities to such an extent as to make the coal suitable for fuel purposes or for coking. If a mixture of equal-sized pieces of coal, slate and iron pyrites be thrown into water having an upward current just strong enough to float off the coal, the heavier pieces of bone, slate and iron pyrites will sink, since they have a greater specific gravity. The floating of a particle depends not only upon its specific gravity but also upon its size and shape, and if the particles of slate and pyrites are smaller than those of the coal, part of the impurities will float away with the coal. The specific gravity of pure bituminous coal is about 1.28, while the specific gravity of commercially pure coal is from 1.30 to 1.35. In washing coal it is the usual practice to save the material having a specific gravity of 1.35 and less, and to reject as waste all material having a specific gravity greater than 1.35. The more nearly uniform the particles of coal and waste the more efficient the process of washing.

A complete coal washing plant comprises three separate processes: (1) the preparation of the coal; (2) the coal washing or mechanical separation of the coal from its impurities; (3) the recovery or separation of the washed coal and the refuse from the water.

Preparation of the Coal.—The preparation of the coal consists in crushing it to the desired size, and the proper screening to subdivide the crushed products into the required number of sizes to facilitate the washing and the selling of the product; or in a plant preparing coal for coking the reduction of all the coal to a size of ½ in. or less. Coal is usually crushed in two stages, the first or coarse crusher reducing the

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coal to nut size or greater, and second the crushers or pulverizers which reduce the coal almost to a powder. The coal is fed into the crusher by means of an automatic mechanical feeder.

Types of Washers.—Coal washers may be divided into four classes: (1) trough washers; (2) continuous ascending current washers; (3) intermittent ascending current washers, and (4) bumping tables.

1. *Trough Washers.*—A trough washer consists of a long trough having low dams or riffles at intervals along its length. The coal is fed in at the upper end of the trough and is carried down to the lower end, the pieces of bone, slate and pyrites being deposited back of the riffles. Trough washers are relatively inexpensive to install, but unless supplied with automatic appliances the operating cost is high.

2. *Continuous Ascending Current Washers.*—In a continuous ascending current washer the coal is fed into the top of an inverted cone, while the water is admitted to the bottom of the cone with a velocity sufficient to carry the particles of coal over the top, while the heavier impurities settle to the bottom of the cone and are carried away. The Jeffrey-Robison inverted cone washer is of this type.

3. *Intermittent Ascending Current Washers.*—Coal washers of the intermittent ascending current type are known as jigs and may be divided into (*a*) piston jigs, in which the movement of the water is produced by pistons, and (*b*) pan jigs, in which the pan in which the coal is placed is moved up and down in the water. The Luhrig jig is of the piston type. The essential part of the Luhrig process is that the coal be graded accurately to size. The jig consists of a rectangular box with hopper bottom, divided about half way down from the top by a partition. In one of the compartments a rectangular piston, operated by an eccentric, works up and down, while the other compartment is closed at the top by a slightly inclined screen plate. The entire jig is filled with water and the movement of the piston gives a pulsating motion which forces the water up and down through the screen plate. The coal to be washed is received on the screen plate near the partition, the action of the water carrying the coal off the top of the jig, while the refuse settles on the screen. The refuse is permitted to flow off the screen and through a valve into a compartment from which it is removed to the refuse heap. The bed of refuse lying on the screen prevents the finer particles of coal from being drawn through the screen. With fine coal an artificial layer of feldspar is carried on the screen.

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The Stewart jig is of the pan or movable screen type. The unwashed coal is admitted through a sliding gate into the jig box which extends below the water level. The bottom of the jig box is composed of perforated screen plates. The coal is carried over the end of the jig, while the waste drops through a gate into the refuse compartment. With the Stewart jig as a general rule no attempt is made to differentiate the sizes of coal, all the coal from nut size down being delivered to the washer. The Stewart jig is very efficient for the coarser sizes of coal, but is not so satisfactory for very fine coal.

4. *Bumping Tables.*—In the Campbell washing table a shallow rectangular box is suspended from above by four rods attached to the screens so as to permit a longitudinal swinging motion of six to eight inches. The bottom of the washer is made in two parts, the lower part being formed by a steel plate, while above this is a false bottom with a $1\frac{1}{2}$ in. space between it and the true bottom. When in operation the coal to be washed is fed on the middle of the table while water is admitted through pipes. The current of water washes the lighter coal upwards and discharges it over the side, while the slate and pyrites settle towards the bottom of the table and are caught by riffles. The table is swung backwards and forwards and is suddenly stopped by a bumper, which moves the coal, and also moves the impurities forward to a point where they are discharged into the refuse compartment.

After the coal has been washed it is dried by being run over screens, and is then stored in bins or is taken direct to the coking ovens.

Centrifugal Jigs.—The impurities may also be separated from the coal without the use of water by means of spiral or other centrifugal separators or jigs.

Screening Coal.—In washing coal for coking, the screens are usually all of such a mesh that nothing over 1 in. will go into the washing machine. Where the impurities are finely distributed in the coal a $\frac{1}{2}$ in. screen may be used. Both revolving and shaking screens are used. Revolving screens are either cylindrical or are shaped like a truncated cone, and are made of bent perforated plates attached to a frame with a central shaft, or of woven wire. Two or more screens may be attached to one shaft in order to screen several sizes of coal. Shaking screens are either suspended from overhead rods or are carried on rollers, the shaking motion being given by eccentrics. Where the impurities are difficult to remove hydraulic sizers have been used with considerable success.

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Operation of a Coal Washer.—The mine run coal is first crushed to sizes of nut coal and under, and is deposited in a raw coal bin. From the raw coal bin the coal is delivered by automatic feeders to an elevator which supplies the coal to one or more revolving screens which separate it into the desired sizes. From the screens the crushed coal is conveyed to the coal jigs where the impurities are removed. The washed coal is sluiced into the draining screens, from which it is deposited by gravity or is elevated by drainage elevators to the shipping pockets The refuse from the coal jigs is carried by conveyors to the refuse bin.

Design of Coal Washers.—Coal washers must be designed to support the necessary screens, crushers, jigs, conveyors, etc., the exact weights of which should be calculated. Great care should be taken to provide sufficient bracing and to use short members in order that the structure may be rigid. The same stresses and specifications should be used for coal washers as for coal tipples (see Appendix I). Ample room should be provided for the jigs and adequate draining provided for the floors. The washery should be well lighted and heated.

The operation of coal washers will be shown by describing an anthracite coal washer and a bituminous coal washer.

The Capouse Coal Washer.*—Detail plans of the Capouse washery are shown in Fig. 207 to Fig. 209. The material is brought from the culm bank by an endless conveyor which delivers the fine material at the foot of the conveyor to the washer. The washer building is 53 ft. by 65 ft. at the foundation, is 80 ft. high, and has a capacity of 120 tons of prepared coal per hour.

The preparation of the coal at the Capouse washery begins at a point where the scraper-line discharges into a chute leading to the main elevator. A man stationed here throws out large lumps, breaks those containing coal into smaller pieces, which are then thrown into the elevator-boot, *A* (see Fig. 208), removes any pieces of foreign material—wood, iron, slate, etc., and controls the feed of the coal to the elevator. In the washery, the main elevator is 65 ft. long from center to center of sprocket-wheels, and carries 71 water-tight buckets, each 12 by 28 in. in size. The elevated material is discharged into a chute, *B*, which feeds the first shaking-screen, *C*. (In washeries the revolving, circular screen has been almost entirely superseded by those of the

* From a paper by George W. Harris, Trans. Am. Inst. Min. Eng., Vol. 36, 1905.

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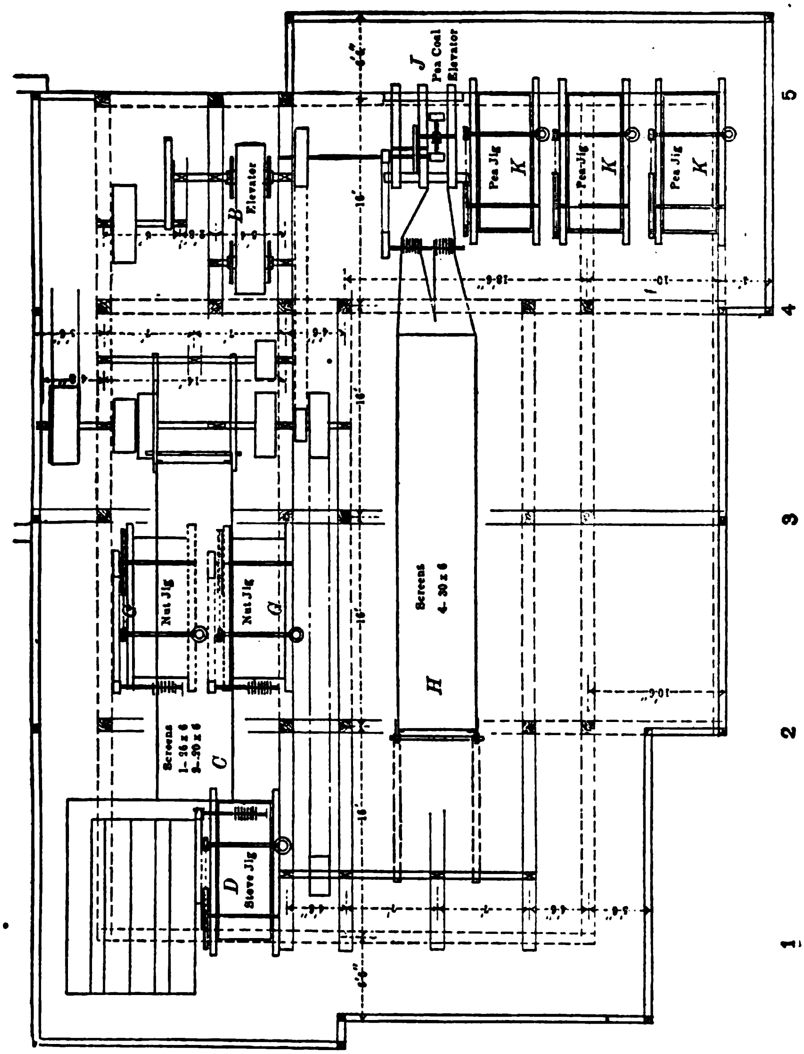

FIG. 207. PLAN OF CAPOUSE WASHERY.

flat shaking type because the fine mesh of the former becomes clogged with dirt, despite all efforts to prevent it.)

The shaker or "mud" screen, *C*, consists of three screens, the top one being 27 by 6 ft., and the two others each 20 by 6 ft. in area. The screens are driven by eccentrics, set so that each one receives a thrust at a different time from the others, an arrangement which is necessary

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in order to avoid undue vibration of the framing. As soon as the material strikes the top screen, it is sprayed with water from a perforated pipe; and, passing down the screen, goes under a box from which a copious stream of water overflows. The first 21 ft. of the top screen have 1.5 in. round holes, through which pass chestnut and smaller size pieces to the screen below. Next to the 1.5 in. round holes

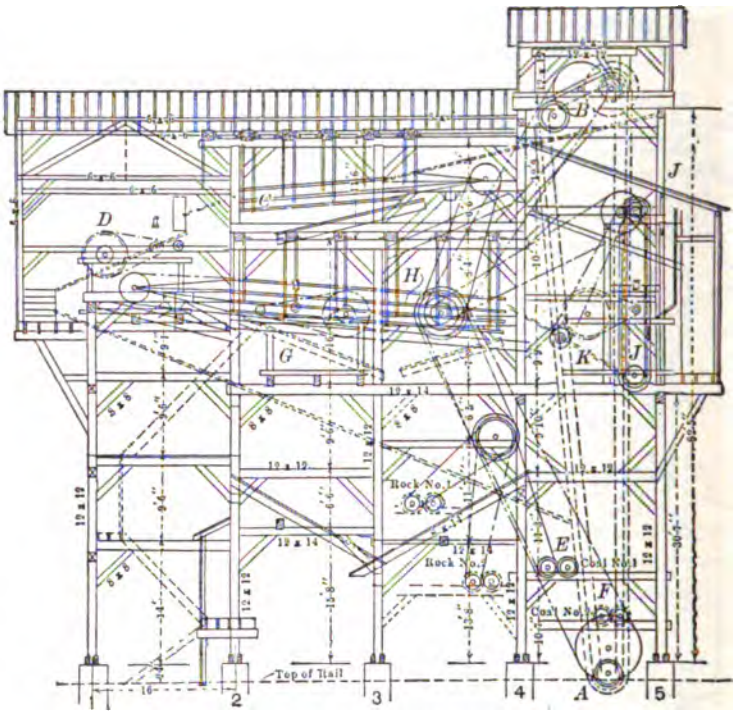

FIG. 208. LONGITUDINAL SECTION OF CAPOUSE WASHERY.

are placed angle-iron, having the angle uppermost, thus, ∧, the edges spaced 0.75 in. apart, which allows flat pieces of slate to fall through. The last 4 ft. of the top screen have 2 in. round holes, which permit pieces the size of stove coal to fall through to a chute; pieces larger than 2 in. pass over the end of the screen to another chute. The stove coal goes to jigs, *D*, thence to rolls, *E* and *F*, and, after being broken to pea size and smaller, to the main elevator-boot, *A*.

The large coal is hand-picked by six men and boys and the slate removed, after which it is sent to the rolls and the main elevators. The

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coal that drops through the top screen of the shaker, *C,* falls on the second screen having $\frac{7}{8}$ in. holes, the chestnut-size passing over, and pea and smaller sizes dropping through the lowest screen. The chestnut-size goes to jigs, *G,* thence to rolls, *E* and *F.* The bottom screen has $\frac{3}{32}$ in. holes which permit the fine coal dirt, slush or culm (it is called by all three names) and mud to pass through to a trough which delivers to a settling-pond near the washery where the stream spreads over a large, nearly level area and deposits the suspended materials. The pea-size and smaller sizes of coal pass over the bottom screen to a second shaker, *H.*

The shaking-screen is simple in construction, effective in action, occupies little space and needs few repairs. At the Capouse washery, each screen is suspended by $\frac{7}{8}''$ by 8″ ash boards, their upper ends being bolted to overhead beams and the lower ends to castings which journal on bars passing under and supporting the screens. Two boards, comprising the hanger on each side, are set at an angle from the vertical so that they act as braces and prevent the screens from swaying sideways. When suspended by rods, the screen travels between guides to insure greater steadiness. Both methods of suspension are used, the boards having the preference.

The shaker, *H,* consists of four tiers of screens, the top one having an area of 30 by 6 ft., the next lower 26 by 6 ft., and the two lower being 20 ft. long by 6 ft. wide. The coal passing through this shaker is separated into sizes as follows: the top screen with $\frac{1}{2}$ in. mesh allows the pea coal to pass over, and No. 1 buckwheat and all smaller to drop through; the next screen, with $\frac{3}{8}$ in. mesh, separates No. 1 buckwheat from the smaller sizes, the latter dropping through to the next lower screen with a $\frac{1}{4}$ in. mesh; this last screen makes No. 2 buckwheat or rice-coal; No. 3 buckwheat or barley passes over the lowest screen which has $\frac{3}{32}$ in. mesh, through which drops the fine coal to be carried by the wash water to settling pond No. 2. The wisdom of keeping the mud from the shaker, *C,* separate from the fine coal of the shaker, *H,* when possible, will be more apparent as time goes on, and this fuel becomes valuable as material for briquettes or for burning as dust.

The 3 buckwheat sizes from the shaker, *H,* go direct to pockets, but the pea coal must be cleaned of slate. After the pea-size leaves the top screen it passes down a chute, in which is a triangular device raised about $\frac{1}{2}$ in. above the bottom, so as to allow flat pieces of slate to pass under while the coal goes to the elevator, *J,* and so to the jigs, *K.*

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After leaving the jigs, the coal passes through a Pardee spiral picker for further cleaning and thence to a pocket.

A number of features about this washery are deserving of special mention. The six jigs (of the Christ type and measuring 11 ft. $\times$ 5 ft. 4 in. $\times$ 6 ft. 9 in.) are driven by a 7×8 in. engine. The coal receives a reciprocating motion in a pan immersed in water, which action causes the slate to sink, while the lighter coal passes out at the top. Generally three jigs are sufficient to clean the coal, the others being held in

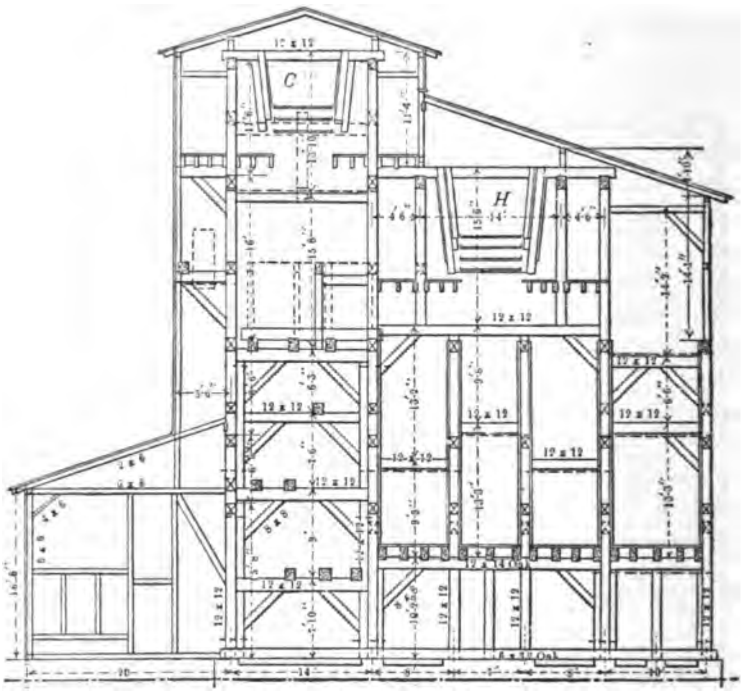

FIG. 209. TRANSVERSE SECTION OF CAPOUSE WASHERY.

reserve. The larger mesh screens on the shakers are of steel, but those of $\frac{3}{32}$ in. mesh, on account of the small perforation, must necessarily have thin metal to prevent clogging, and therefore are made of bronze, in order better to withstand the action of the acid mine water used in washing the coal. At the Capouse washery, the shaker, *H*, has, above the tier of screens, four overflow water-boxes, which are very efficient.

The shaker, *C*, receives from 165 to 170 thrusts per minute and the

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shaker, *H,* about 180. To re-break large coal to pea-size and smaller, two rolls are used, the roll, *E,* being 24 × 24 in., and *F,* 21 × 24 in. The two rock-rolls shown in Fig. 208 were used to crush slate, taken from the coal, to a size not larger than pea, which was then sent with the fine coal to fill old mine workings, a method since abandoned at the Capouse washery.

A large quantity of water is most important for successful washing, the supply for the Capouse washery being obtained from an adjacent shaft. A bull-pump raises water from the mines and delivers it to a reservoir near the washery, the pumping equipment of which consists of 3 No. 10 Knowles pumps, 16 × 10 × 16 in. It is estimated that each pump furnished 500 gal. per min.; two pumps raise water to the top of the washery through their two 6 in. columns which connect at the top; the third pump supplies water for flushing the material into conveyors at the culm bank. All water pipes about the plant are at such an angle that when work stops, valves are opened and the water drains out, thus preventing the pipes from freezing up in cold weather.

The machinery is operated by a 24 × 24 in. Dickson engine, running at 78 revolutions per minute. The steam is supplied by three fire-tube boilers, housed in a substantial brick building, situated at a distance of about 500 ft. from the washery. The labor required to run the plant consists of from 40 to 50 men and boys; a larger number being needed in winter than in summer to clean out railroad cars, etc. The outside force includes a foreman in charge of the whole plant, 2 hosemen, 10 or 12 men on conveyor lines, 2 men to run the conveyor engines, 1 man at the elevator boot, and 2 men at the settling-ponds. The force inside the washery is distributed as follows: 1 machinist or oiler, 1 carpenter or repair-man, 3 jig-runners, 8 slate pickers, 1 engineer, 1 man at the head of the main elevator, 4 loaders, 2 car repair men, 6 slate dump men. In addition, there are at the boiler-house 3 firemen, 1 man to cart out ashes and one man to wheel in coal. The Capouse plant has shipped coal since November 1, 1900, and during 1904 worked 182½ days of 9 hours each, producing 184,004.7 tons. The record for the washery was made in January, 1902, when 31,018 tons were produced in 262½ hours.

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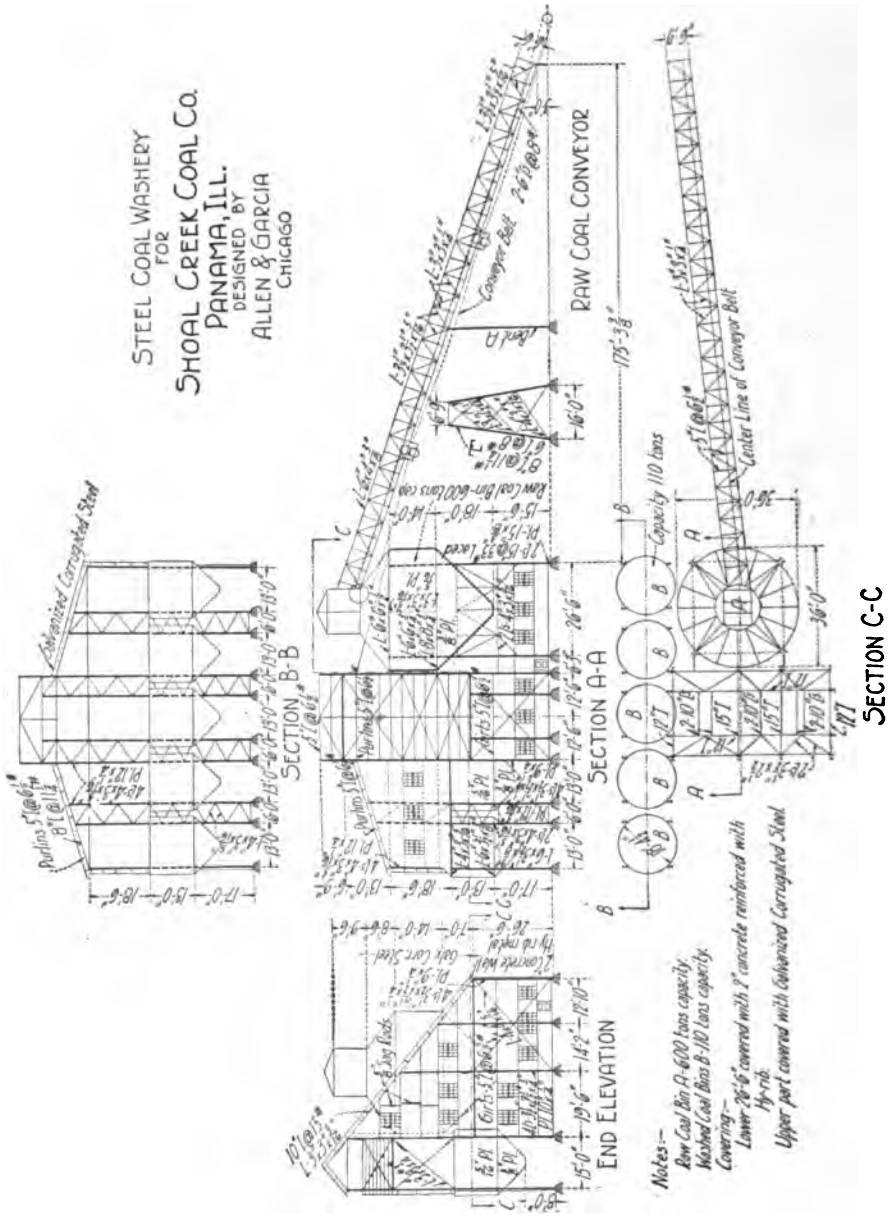
STEEL COAL WASHERY FOR SHOAL CREEK COAL CO. PANAMA, ILL. DESIGNED BY ALLEN & GARCIA CHICAGO
RAW COAL CONVEYOR
SECTION B-B
SECTION A-A
SECTION C-C
END ELEVATION
Notes:—
Raw Coal Bin A-600 tons capacity.
Washed Coal Bins B-110 tons capacity.
Covering—
Lower 26'6" covered with 2" concrete reinforced with Hy-rib.
Upper part covered with Galvanized Corrugated Steel.

FIG. 210.

Steel Coal Washery for Shoal Creek Coal Co.—The steel coal washery for the Shoal Creek Coal Co., Panama, Ill., designed by the Allen & Garcia Co., Chicago, Ill., and built by the Wisconsin Bridge and Iron Co., is shown in Fig 210. The coal is brought into the washery by a belt conveyor and is dumped into the raw coal bin, *A*, having a capacity of 600 tons. The coal is drawn from the raw coal bin and is carried by conveyors to the crushers and jigs. The washed coal is stored in the washed coal bins *B*, which have a capacity of 110 tons each. The lower 26 ft. 6 in. of the building is covered with a 2 in. concrete wall reinforced with hy-rib expanded metal, while the upper part of the building and the roof is covered with No. 20 galvanized corrugated steel fastened to channel purlins and girts. The main cylinders of the bins are made of $\frac{5}{16}$ in. steel plates, while the hoppers are made of $\frac{3}{8}$ in. steel plates.

The shipping weight of the structural steel was 425,000 lbs.; while the weight of the corrugated steel was 24,000 lbs.

The steel plates in the bins are protected on the inside with two coats of coal tar paint, applied to the steel plates after they were thoroughly cleaned in the field. Provision was also made for lining the bins with concrete if it should become necessary.

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CHAPTER XII.

THE DESIGN OF COAL BREAKERS

Introduction.—Anthracite coal comes from the mine in lumps and also contains more or less slate and other impurities. To burn freely anthracite must be broken to nearly uniform sizes so that there will be a free passage of air through the coal. The preparation of anthracite coal consists in removing the slate and other impurities, and in breaking and grading the coal to the proper size. The common market sizes of anthracite coal are given in Table XXVII a The allowable amount of impurities in anthracite varies from 5 per cent for steamboat or broken to 12 to 15 per cent for the finer grades of coal.

TABLE XXVII a.

SIZES OF ANTHRACITE COAL.

Name.	Diameter of Holes over Which must Pass, In	Name.	Diameter of Holes over Which must Pass, In.
Steamboat.......	$4\frac{1}{2}$	Pea	$\frac{1}{2}$ to $\frac{5}{8}$
Broken...........	$3\frac{1}{4}$	No. 1 Buckwheat	$\frac{1}{4}$ to $\frac{5}{16}$
Egg	$2\frac{1}{4}$ to $2\frac{5}{16}$	No. 2 Buckwheat	$\frac{3}{16}$
Stove.	$1\frac{1}{2}$ to $1\frac{5}{8}$	No. 3 Buckwheat	$\frac{3}{32}$ to $\frac{1}{8}$
Chestnut..........	$\frac{7}{8}$ to $1\frac{5}{16}$	Culm, through	$\frac{3}{32}$ to $\frac{1}{8}$

The slopes of chutes lined with steel plates down which coal will slide are given in Table XXVII b. If the coal is to start on the chute the pitch should be increased about 25 per cent.

TABLE XXVII b.

SLOPES OF CHUTES LINED WITH STEEL DOWN WHICH COAL WILL SLIDE.

Size of Coal.	Pitch in Inches per Foot.	
	Dry Coal.	Wet Coal.
Run of Mine..	5	
Lump and Steamboat................................	4	
Broken...	4	$2\frac{1}{2}$
Egg and Stove......................................	4	$2\frac{1}{2}$
Chestnut and Pea...................................	6	4
Buckwheat..	9	7
Culm...		9

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In preparing anthracite the lump coal is run over screens to separate the lump coal from the fine coal; the lump coal is run through crushers and rolls, and is run over screens. The slate, bone and other impurities are separated from the coal by jigs or spiral separators.

A diagrammatic scheme for handling the coal in the Lehigh Coal and Navigation Company's breaker is shown in Fig. 211.

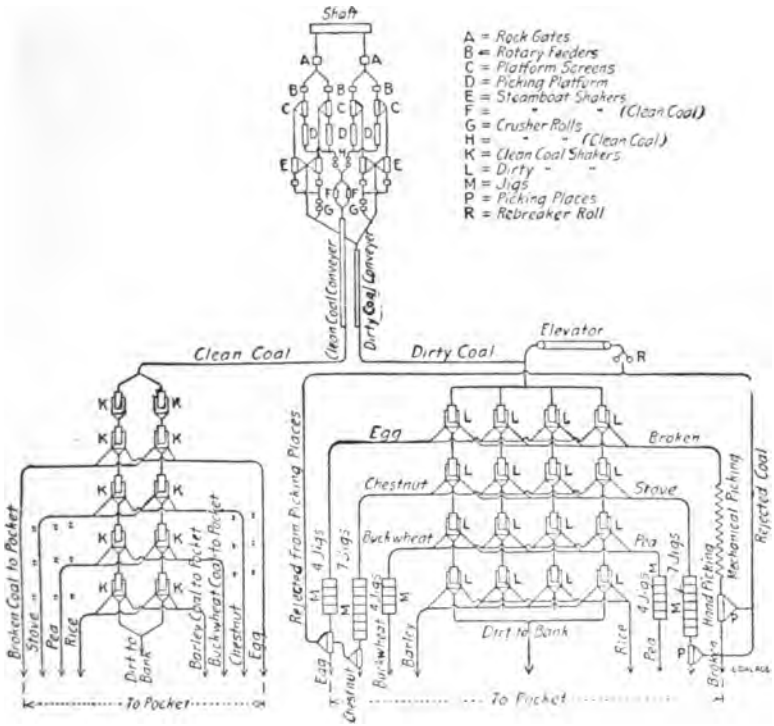

FIG. 211. DIAGRAMMATIC SCHEME FOR HANDLING COAL IN THE BREAKER OF THE LEHIGH VALLEY COAL AND NAVIGATION COMPANY.

Jigs and Spirals.—Jigs similar to those used in coal washers are sometimes used in breakers. The large pieces of impurities are hand picked, while the impurities are usually separated from the fine coal in mechanical or spiral separators.

Design of Coal Breakers.—Coal breakers carry shaking screens, crushers, rolls, shafting, etc., which loads must be carefully calculated

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and be provided for. Great care must be used to make the structure rigid and to reduce to a minimum the vibrations due to the shaking screens, crushers and other equipment. The problem is similar to the design of coal tipples and coal washers which have already been considered. The same stresses and specifications should be used for coal breakers as for coal tipples.

The operation of a coal breaker will be illustrated by describing two coal breakers in detail.

The Coaldale Breaker, Lehigh Valley Coal and Navigation Company.*—The plant consists of a steel coal tipple and a steel breaker building, and has a capacity of about 3,500 tons of coal per day, with 118 men, at a cost of 10 to 12 cts. per ton.

The scheme of preparation is shown in Fig. 211 and is as follows:

Preparation of the Coal in the Coal Tipple.—A disposition of the mine rock is made in the head house or coal tipple, when necessary, at the dump by means of a by-pass gate *A* in the bottom of the dump chute. The coal stream from the dump is passed over the platform shakers *C*, having 6 in. round mesh, and a consequent separation made into two streams, one of lump size, the other of mixed steamboat and smaller sizes. Following first the lump coal stream, it is seen to descend to the picking tables *D*, which are about 20 ft. long and have a pitch of 2″ to 2½″ in 12 in. Here all the rock is removed and the cleaned lump coal which passes off is broken down in the crushers *H* to steamboat size and smaller. Steamboat size is removed by the shakers *F*, which have a 4½ in. round mesh, and is subsequently rebroken. All of this stream goes to that one of the two conveyors which is devoted to cleaned coal.

Following next the second stream, which consists of everything passing through the 6 in. mesh of the platform screens, it is found to be led over the shakers *E*, having a 4½ in. round mesh. The steamboat coal thus made is hand-picked, then crushed and taken to the clean-coal conveyor. Everything passing through the mesh of the shakers *E* is led directly to what is known as the "dirty" coal conveyor, to distinguish it from its partner.

Preparation of the Coal in the Breaker.—There are thus two streams of coal passing up to the head of the breaker; one cleaned, the other not cleaned, and both a mixture of broken and all smaller sizes, except when it is desired to ship steamboat size, which is done by

* Coal Age, Oct. 28, 1911.

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omitting to break down that size in the head house. The clean-coal stream is usually sized over two sets of shaking screens similar to those listed in Table XXVII c, and is passed from them directly to the pockets with more or less examination in the chutes.

The dirty-coal stream is sized over four sets of shakers, or as a rule, double the number used for the clean coal, similar to those listed in Table XXVII c. Broken size from these screens is usually cleaned by spiral pickers or some other mechanical device, supplemented more or less by hand-picking; occasionally it is jigged. Egg, stove, chestnut and pea sizes are usually led directly from the screens to the jigs. It is the practice in some breakers, however, to spiral or otherwise mechanically pick these sizes on their way to the jigs, and it depends a great deal on local conditions whether or not a profitable percentage of pure coal can thus be deflected to the pockets. Buckwheat size is jigged in a number of cases, but more often it passes directly from the screens to the pocket, as do also the rice and barley sizes.

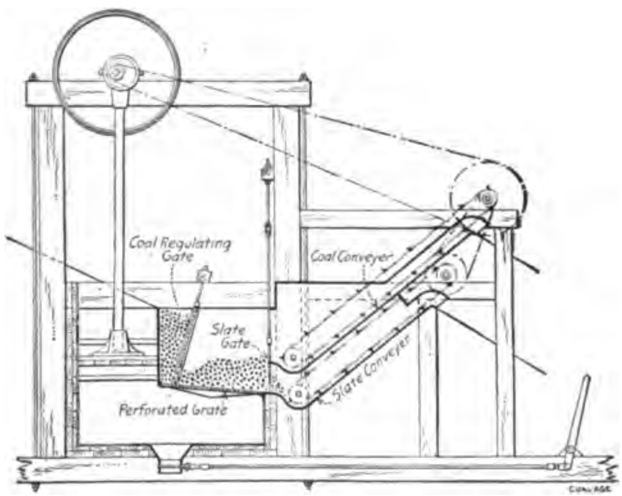

FIG. 212. "LEHIGH VALLEY" TYPE OF PLUNGER JIG.

Table XXVII e gives data relating to jigs of the "Lehigh Valley" plunger type, shown in Fig. 212, which are replacing other styles in a good many instances. The jigged coal is thoroughly examined and occasionally resized before passing to the pockets. The usual provisions are made for breaking down the material rejected at the jigs and various picking places, and for resizing and cleaning the same.

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TABLE XXVII c.

PURE COAL SCREENS, LEHIGH VALLEY COAL AND NAVIGATION COMPANY'S BREAKER.

Number.	Size of Coal Passing Over.	Round Mesh, Diameter, In.	Total Area, Sq. Ft.	Tons per Hour.	Tons per Sq. Ft. per Hour.	Revolutions of Cam Shaft.
2	Broken	3¼	90	6.28	0.070	150 to 165
2	Egg..........	2 5/16	90	7.08	0.078	
2	Stove	1 5/8	90	10.97	0.122	
2	Nut..........	7/8	90	17.25	0.192	
2	Pea..........	9/16	120	9.94	0.083	
2	Buck.........	5/16	120	13.20	0.110	
2	Rice.........	3/16	120	10.57	0.088	
2	Barley.......	3/32	120	4.21	0.035	

TABLE XXVII d.

MAIN SCREENS, LEHIGH VALLEY COAL AND NAVIGATION COMPANY'S BREAKER.

Number.	Size of Coal Passing Over.	Round Mesh, Diameter, In.	Total Area, Sq. Ft.	Tons per Hour.	Tons per Sq. Ft. per Hour.	Revolutions of Cam Shaft.
4	Broken	3¼	180	25.12	0.139	150 to 165
4	Egg..........	2 5/16	180	28.30	0.157	
4	Stove........	1 5/8	180	43.88	0.244	
4	Nut..........	7/8	180	69.01	0.383	
4	Pea	9/16	240	39.75	0.165	
4	Buck.........	5/16	240	52.78	0.220	
4	Rice..........	3/16	240	42.30	0.176	
4	Barley	3/32	240	16.86	0.070	

TABLE XXVII e.

JIGS, LEHIGH VALLEY COAL AND NAVIGATION COMPANY'S BREAKER.

Number.	Size of Coal.	Size Perforations in Grates, Diameter, In.	Total Grate Area, Sq. Ft.	Revolutions Plunger Shaft.	Tons per Hour.	Tons per Hour per Sq. Ft. of Grate Surface.
4	Egg..........	7/8 and 5/8	64	85	28.3	0.442
7	Stove........	7/8 and 5/8	112	85	43.88	0.392
7	Nut	5/8 and 1/2	112	90	69.01	0.616
5	Pea	3/8 and 5/16	80	100	39.75	0.497
4	Buck.........	3/8 and 5/16	64	100	52.78	0.824

The Cottonwood Coal Breaker.*—The coal breaker was erected in 1903 for the Cottonwood Coal Co., Stockett, Mont. The coal was mixed with sulphur balls, slate and bone, making the problem of cleaning a difficult one. Scarcity of water made it impossible to operate a coal washer. The specific gravity of the material mined was: sulphur balls, 4.141; slate, 2.402; bone, 1.963; gray coal, 1.50 to 1.751; coking coal, 1.44; other coal, 1.293. After many experiments the coal breaker was constructed.

* From a paper by Lewis Stockett, Trans. Am. Inst. Min. Engs., Vol. XXXV, 1905.

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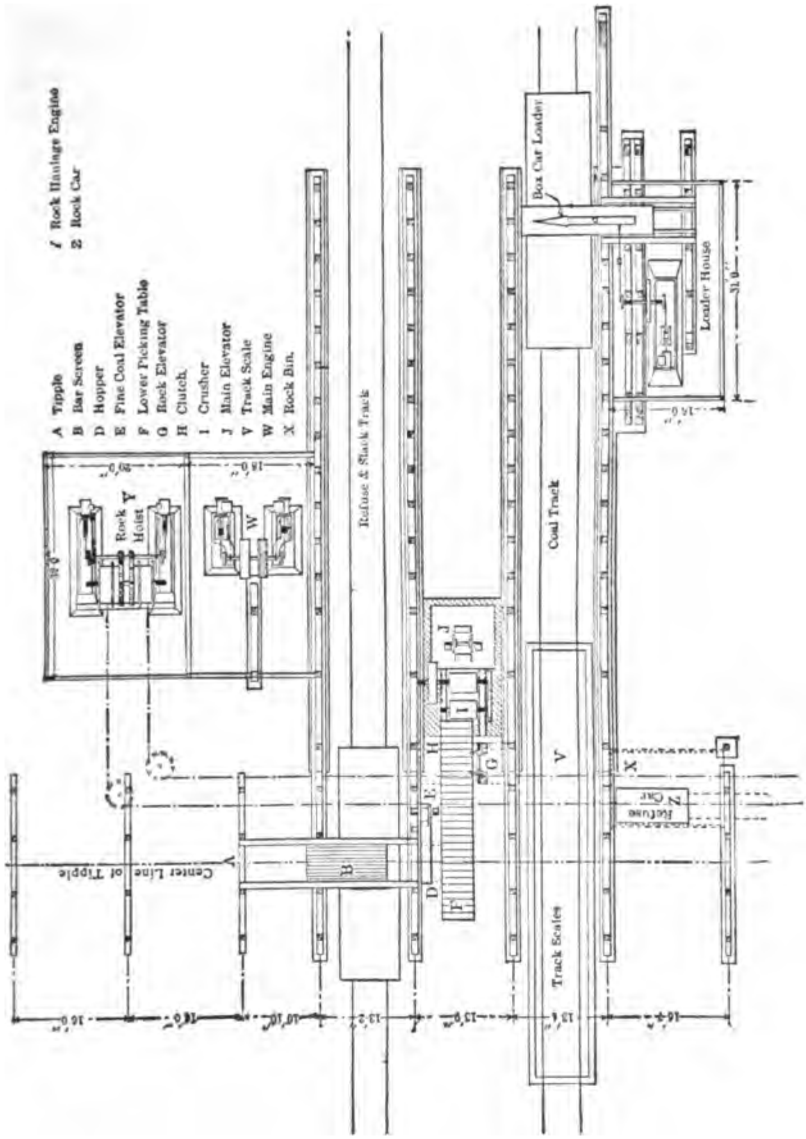

Fig. 213. Plan of Coal Breaker.

The detail plans of the breaker are shown in Fig. 213 to Fig. 217. The same letters are used in all of the figures, so that reference will be made to the letters and not to the figures.

The coal, carried from the mine in pit cars of a capacity averaging about 1.5 tons to the car, is weighed on an automatic scale and thence dumped by the cross-over tipple *A*, over the bar-screen *B*, 12 ft. long, 6 ft. wide and having a pitch of 6 in. to the foot with spaces between

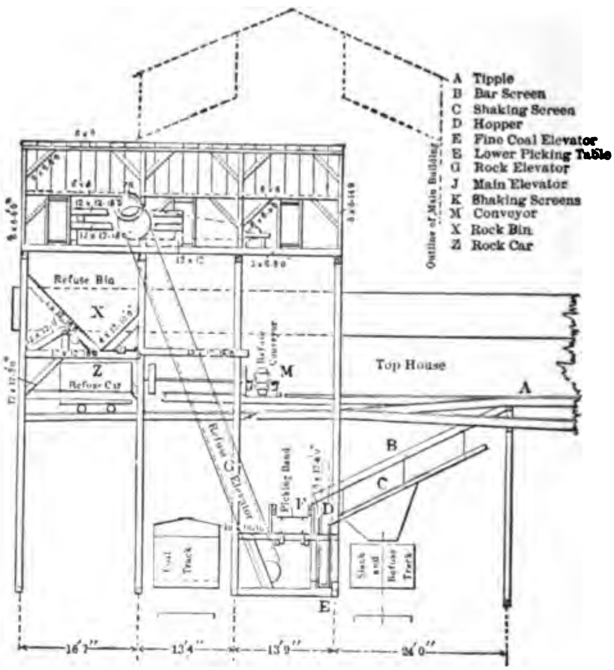

FIG. 214. SECTION OF COAL TIPPLE.

the bars 2 in. wide, which screens out that portion already small enough (about 30 per cent of the total). It then falls on the shaking-screen *C*, having 1 in. round perforations, 6 in. throw and 100 strokes per minute, which removes the slack from the coal. The slack is loaded directly into railroad cars or is taken to the boiler-room by means of a wire-rope conveyor. The portion of the coal that goes over the shaking-screen *C* slides into a hopper *D*, from which it is fed into an elevator *E*, consisting of a rubber belt 16 in. wide, having 8×14 in. buckets placed every 16 in. and operated with a speed of 225 buckets per minute.

The coal that passes over the screen *B* falls upon a traveling belt *F*, 4 ft. 6 in. wide and 26 ft. long, having a speed of 33 ft. per minute,

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and from this belt any large pieces of slate and other impurities are removed by men stationed on the sides and is thrown into a rock-elevator *G*. This traveling belt is operated by a clutch gear *H*, which in case of a very large quantity of impurities appearing is thrown out, the belt stopped and all of the impurities removed before the coal drops into the rollers *I*, which reduce the coal to a size not exceeding a 4 in. cube. It was found necessary to reduce to a 4 in. size in order to prevent the concealment of a sulphur ball in a lump of coal. The rollers

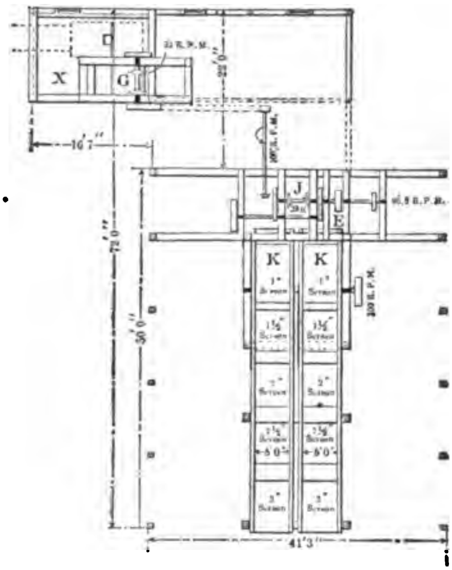
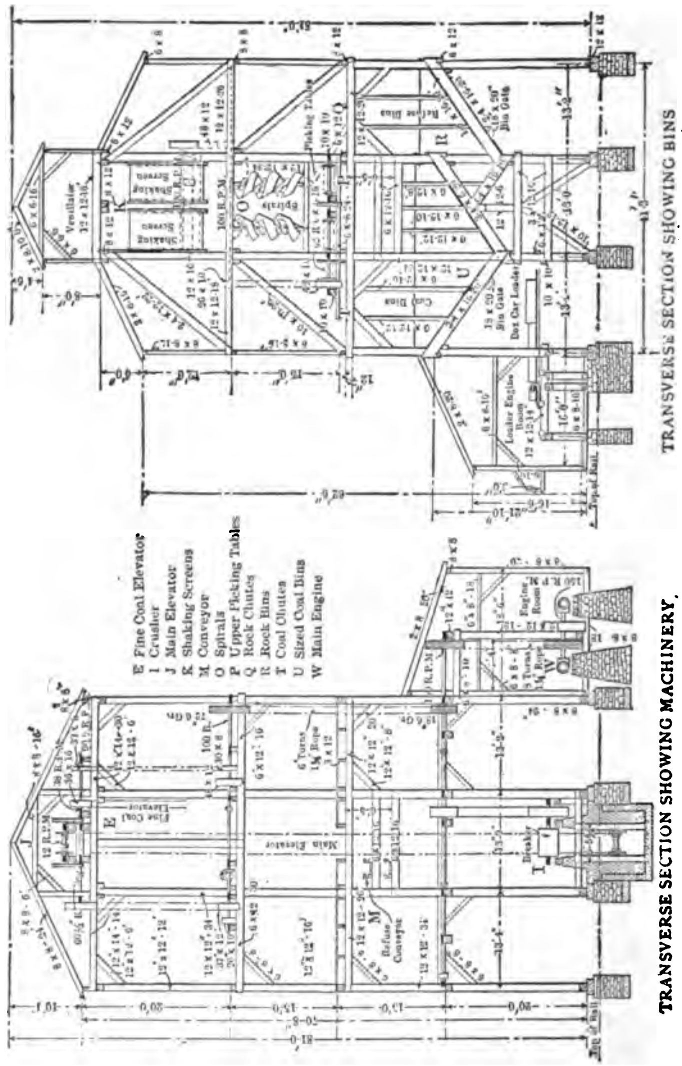


FIG. 215. PLAN OF SCREENS.

are of the removable-tooth style, 36 in. in diameter and 48 in. wide, and revolve at 75 revolutions per minute.

From the rollers the coal is elevated by a continuous elevator *J*, having buckets 12 × 30 in., and operated at a speed of 65 buckets per minute. Each bucket has a capacity of 110 lbs. of coal when level-full, which is equivalent to 200 tons per hour; the capacity of the fine coal elevator *E* is 90 tons per hour, giving a combined elevating capacity of 290 tons per hour, or 2,900 tons per day of 10 hours, an amount which added to the slack screened out at *C* gives a total capacity of 3,200 tons per day.

The coal elevated by the elevators, *E* and *J*, is evenly divided over



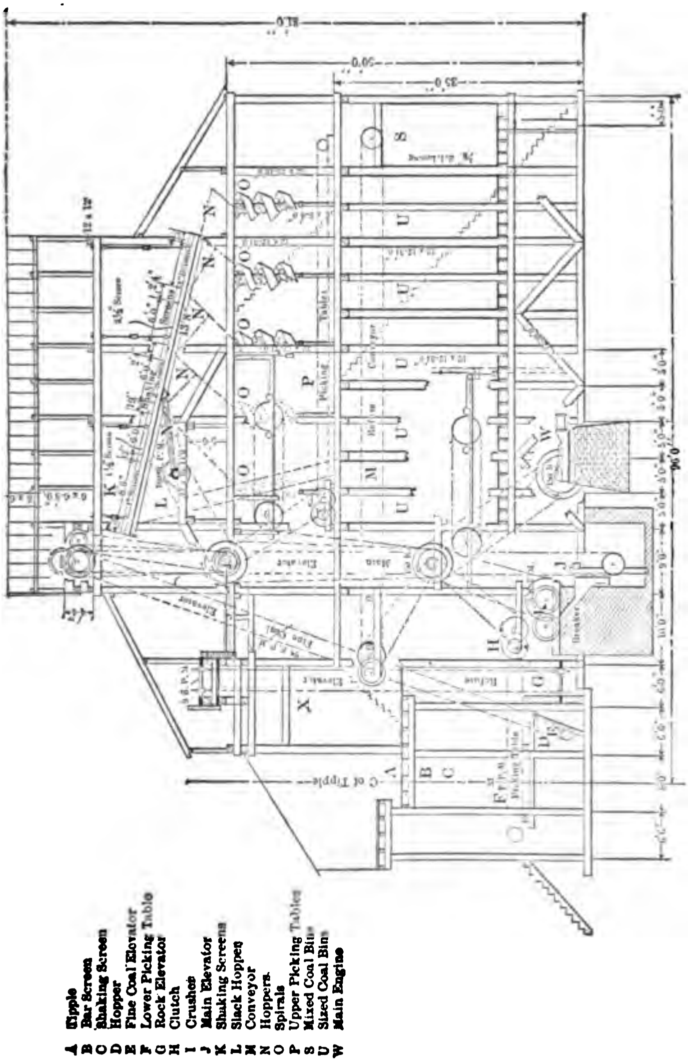
TRANSVERSE SECTION SHOWING BINS.

TRANSVERSE SECTION SHOWING MACHINERY.

FIG. 216. TRANSVERSE SECTIONS OF COAL BREAKER.

the shaking-screens *K*, 5 ft. wide and 46 ft. long, having a 3 in. pitch to the foot, a 6 in. throw and 100 strokes per minute. The plates of the screens have respectively 1, 1.5, 2, 2.5 and 3 in. round perforations, and separate the coal into slack, pea, nut, stove, egg and broken sizes.

The slack resulting from the breakage of the coal is found to be clean, and, not needing any further preparation, it descends through



LONGITUDINAL SECTION SHOWING MACHINERY

FIG. 217. LONGITUDINAL SECTION OF COAL BREAKER.

the hopper *L* to the top strand of the conveyor *M* (having 10 × 20 in. flights spaced every 3 ft.), and is taken directly to the mixed-coal bin *S*. The other sizes are fed by means of the hoppers *N* into spiral separators *O*, which separate the greater part of the impurities from the coal. These impurities pass either to the lower strand of the conveyor *M* and are conveyed to the rock elevator *G*, or from one set of spirals

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to the bins *R* by means of chutes *Q*, which give an opportunity to re-pick the refuse by hand and save any coal that may be in it. This refuse is finally loaded into railroad cars and used by the railroad for widening banks, etc.

From the spirals *O* the coal drops on two picking bands *P*, 4 ft. wide and 50 ft. long and having a speed of 40 ft. per minute, which convey it to the mixed-coal bin *S*, and give an opportunity to pick out by hand any impurities not removed in the spirals; from one set of spirals, inclined chutes *T* pass the coal into bins *U* for loading straight sizes, any remaining impurities being removed by hand while the material is on these chutes.

The rock elevator *G*, having continuous buckets 12×30 in. and a speed of 50 buckets per minute, elevates the impurities into a bin *X*, from which it is loaded into a 6 ton car and hoisted by a pair of geared tail-rope engines, with 10×18 in. cylinders to the top of the adjoining hill and automatically dumped.

The machinery of the entire plant is driven by a double engine *W*, with cylinders 13×18 in., running 150 revolutions per minute, the connection to the first and second line-shafts being made by rope drives, and all other connections by rubber belts.

From the bins *S* and *U* the coal is loaded into railroad cars, a box-car loader being placed opposite the chute *S* to load box-cars. The cars are weighed on the scale *V*, and it may be of interest to know that, when loading the mixed coal into box-cars, the entire product of the breaker passes through an opening 12×14 in. in area.

The building is heated by steam-coils supplied with live steam, and is lighted by incandescent electric lights. It has two stand-pipes with hose connections and hose for fire protection.

On account of the slight difference in the specific gravity of the gray coal and the bone, the spirals are adjusted so as to retain only the slate and flat sulphur balls, leaving the bone to be removed by hand. The round sulphur balls which on account of their shape are the first to leave the spirals and go with the coal, also have to be removed by hand.

The results obtained are given below, showing the percentages of refuse in the various sizes: In pea coal, 4; in nut coal, 3; in stove coal, 3; in egg coal, 2; in broken coal, 1, and in mixed coal, 2 5 per cent. Of 2,000 tons of the mine product, which is daily dumped into the breaker, 200 tons of the various impurities are removed, and these impurities do not contain on an average over 1 per cent of coal.

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The mixed coal is used by the railroads as a locomotive fuel, and proves an excellent article; the various sizes are used in the commercial trade, the slack and pea sizes making the very best of boiler fuel.

The cost of the breaker, in the section where the highest wages are paid in the United States and where freight is a very large item, was $42,517.90, divided as follows:

General expense	$ 472.45	Scale	$ 123.75
Foundations	2,445.46	Hardware	805.91
Lumber	3,706.31	Labor	10,700.82
Machinery	12,764.89	Freight	8,844.13
Separators	2,753.18	Total	$42,517.90

The above includes the cost of 300,000 ft. of lumber and 400 yards of masonry as well as of duplicate pieces for all of the parts of the machinery that are liable to break or wear out.

The cost of operation of the plant over and above that of the force previously used on the old tipple is as follows:

Coal inspector on lower picking band	$ 3.00
3 men on lower picking band	7.50
1 man looking after screens	2.50
1 man looking after spirals	3.00
15 boys picking slate, etc.	15.00
1 breaker boss	3.00
1 engineer, also hoists the rock	4.00
1 machinist, oils and keeps machinery in order	3.50
1 man loading rock	2.50
	$44.00

Cost per ton of 1,500 tons (1,800 less 300 slack)	$ 0.03
To which add interest on investment, taxes, insurance, wear and tear, supplies used and repairs, which as near as can be estimated is per ton	0.03
Total additional cost per ton	$ 0.06

The largest item of cost per ton, however, comes from the decrease of production from what was formerly shipped, by reason of the removal of the impurities, the total cost being now charged to 1,500 tons of lump coal instead of 1,700 tons, as formerly; this is met by the increased price received for the coal, which at the increased price is a cheaper fuel to the consumer than the former product at the lower price.

The success of this plant will make available large fields of coal in Montana, which on account of the impurities present were hitherto regarded as unworkable. However, it is only with coals hard enough to keep their shape, and where a marked difference exists both in the

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specific gravity and the shape of the coal and the impurities, that the spirals will work successfully.

FIG. 218 a. TAYLOR REINFORCED CONCRETE COAL BREAKER.

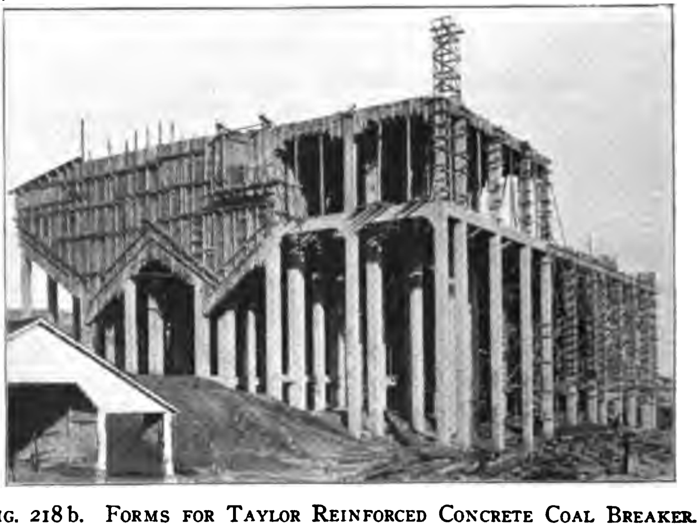

FIG. 218 b. FORMS FOR TAYLOR REINFORCED CONCRETE COAL BREAKER.

Reinforced Concrete Coal Breaker.—The reinforced concrete coal breaker built by the D. L. & W. R. R., at Taylor, Pa., is shown in Figs.

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218 a to 218 d. A general view of the breaker is shown in Fig. 218 a, the forms for the concrete are shown in Fig. 218 b, while sections of the breaker are shown in Fig. 218 c and Fig. 218 d. The breaker is 107

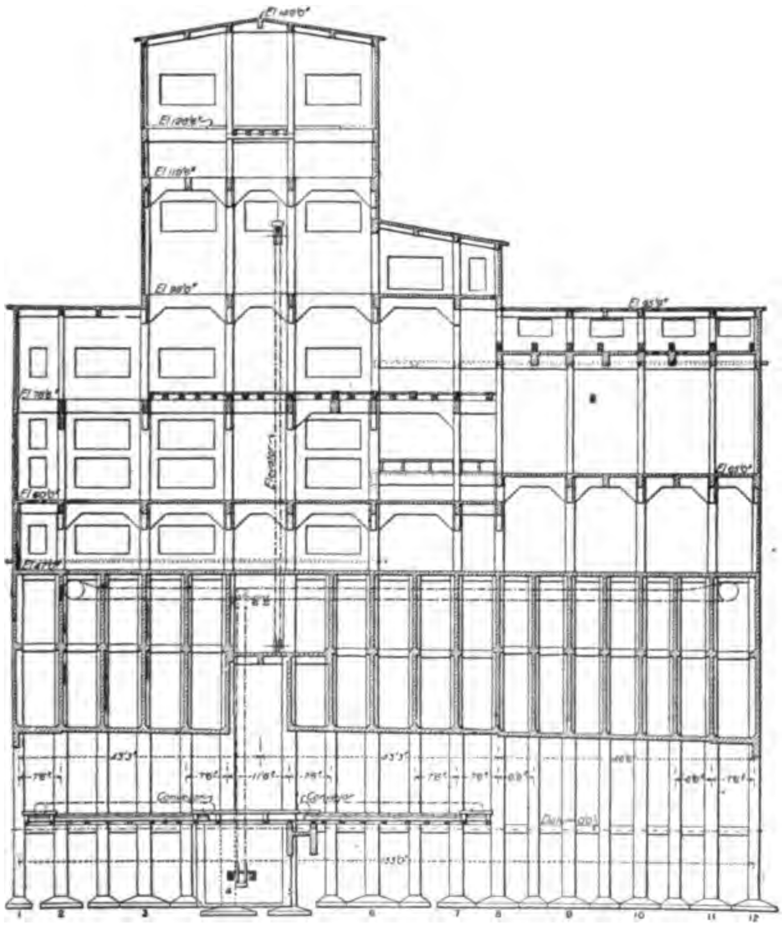

FIG. 218 c. LONGITUDINAL SECTION B–B, TAYLOR REINFORCED CONCRETE BREAKER.

ft. 6 in. wide, 133 ft. long, and 167 ft. from the bottom of the foundations to the top of the roof. All floors carrying machinery were designed for a live load of 200 lbs. per sq. ft. with 100 per cent impact; except where the loads were known, which were used with 100 per

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cent impact added. The roof was designed for a live load of 40 lbs. per sq. ft. Shaking screens weighed from 32,000 to 38,000 lbs.; while the jigs weighed 50,000 lbs. each. In designing the coal pockets the angle of friction between coal and concrete was taken as $\phi' = 27$ degrees.

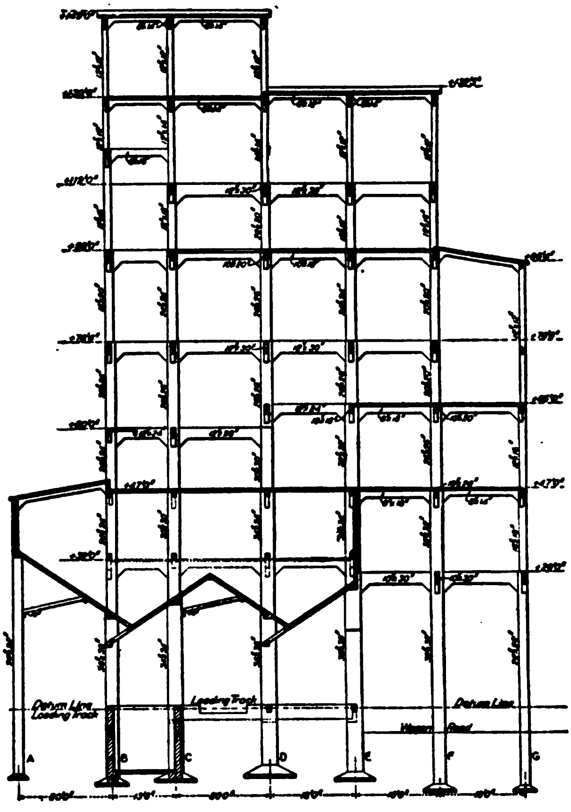

FIG. 218 d. TRANSVERSE SECTION 4-4; TAYLOR REINFORCED CONCRETE BREAKER.

The working unit stresses were: extreme fiber stress in concrete in beams, 500 lbs.; tensile stress in reinforcing steel, 15,000 lbs.; shear on concrete, 50 lbs.; the remainder of the diagonal tension being taken by the stirrups designed for 10,000 lbs. per sq. in. Columns were designed for 400 lbs. per sq. in. on concrete and 10,000 lbs. per sq. in. on longitudinal reinforcing steel. The breaker contains 5,000 cu. yd. of concrete and 500 tons of reinforcing steel.

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For a detailed account of the breaker, see Engineering Record, Dec. 3. 1910, or Proceedings National Association of Cement Users, Vol. VII, pp. 371 to 382.

Principles, formulas, and specifications for the design and construction of reinforced concrete mine structures are given in Appendix III.

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CHAPTER XIII.

Miscellaneous Structures.

Steel Rescreening Plant.—The steel rescreening plant for the Bituminous Coal Washing Co., built by the Wisconsin Bridge and Iron Co., is shown in Fig. 219 a. The conveyor bridge receives the coal at a point 98 ft. 9¾ in. from the building and 10 ft. 8 in. above the foundations of the building. The inclination of the conveyor belt is 16 degrees. The coal is run over shaking screens and through 16 spiral separators. The spiral separators weigh 800 lbs. each. After screening and separating the coal is run into storage bins. The building is covered on the sides with a 2 in. plaster wall reinforced with hy-rib expanded steel. The roof is covered with Federal cement roofing tile. The windows are glazed with wire glass set in steel sash. The bins are made of ¼ in. plate with 6″ @ 8 lb. channel stiffeners. The floors are made of reinforced concrete carried on steel beams.

The weight of the structural steel in this plant is 380,000 lbs. The conveyor gallery is covered with 52 squares of corrugated steel, weighing 7,500 lbs. The cement tile roof cost about $1,000 in place. The 2″ cement plaster walls reinforced with hy-rib expanded metal cost $25.00 per square in place.

Rosiclare Fluorspar Mining and Milling Plant.—The steel mining and milling plant built for the Rosiclare Lead and Fluorspar Mines, Rosiclare, Ill., by the Wisconsin Bridge and Iron Co., is shown in Figs. 219 b to 219 e. The fluorspar, a compound of calcium and fluorine, CaF_2, occurs in combination with lead and zinc and also with calcite and quartz, which must be removed before the fluorspar can be used as a flux in blast furnaces and in basic open-hearth steel furnaces, or for other commercial uses.

The mine has three compartments, two for hoisting, 5 ft. 5 in. × 4 ft. 4 in., and one for pumps, 5 ft. 5 in. × 3 ft. 8 in. The steel head frame is 84 ft. 6 in. from the collar of the shaft to the center of the sheaves, which are 5 ft. in diameter. The hoist is placed 27 ft. from the center of the shaft. The hoisting ropes are 1 in. in diameter. The

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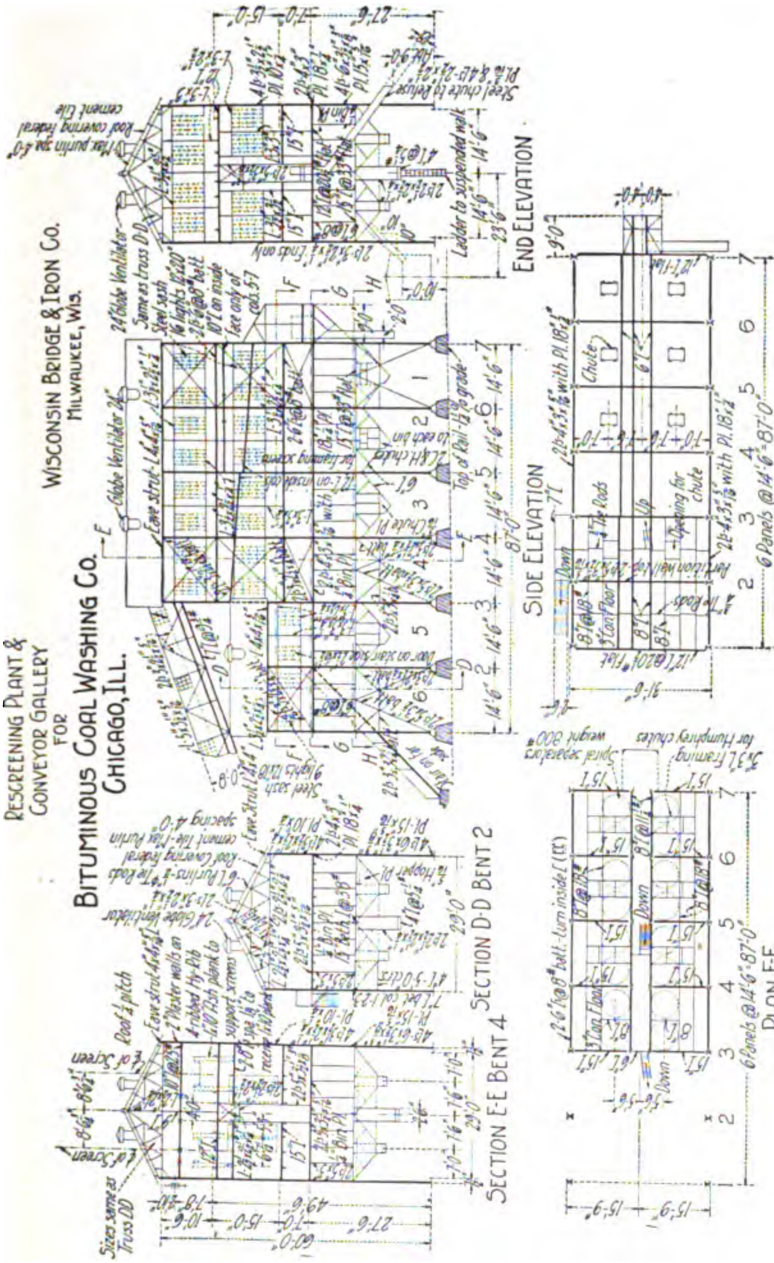
RESCREENING PLANT & CONVEYOR GALLERY
FOR
BITUMINOUS COAL WASHING CO.
CHICAGO, ILL.
WISCONSIN BRIDGE & IRON CO.
MILWAUKEE, WIS.
SIDE ELEVATION
END ELEVATION
SECTION D-D BENT 2
SECTION E-E BENT 4
PLAN F-F
PLAN G-G

FIG. 219 a.

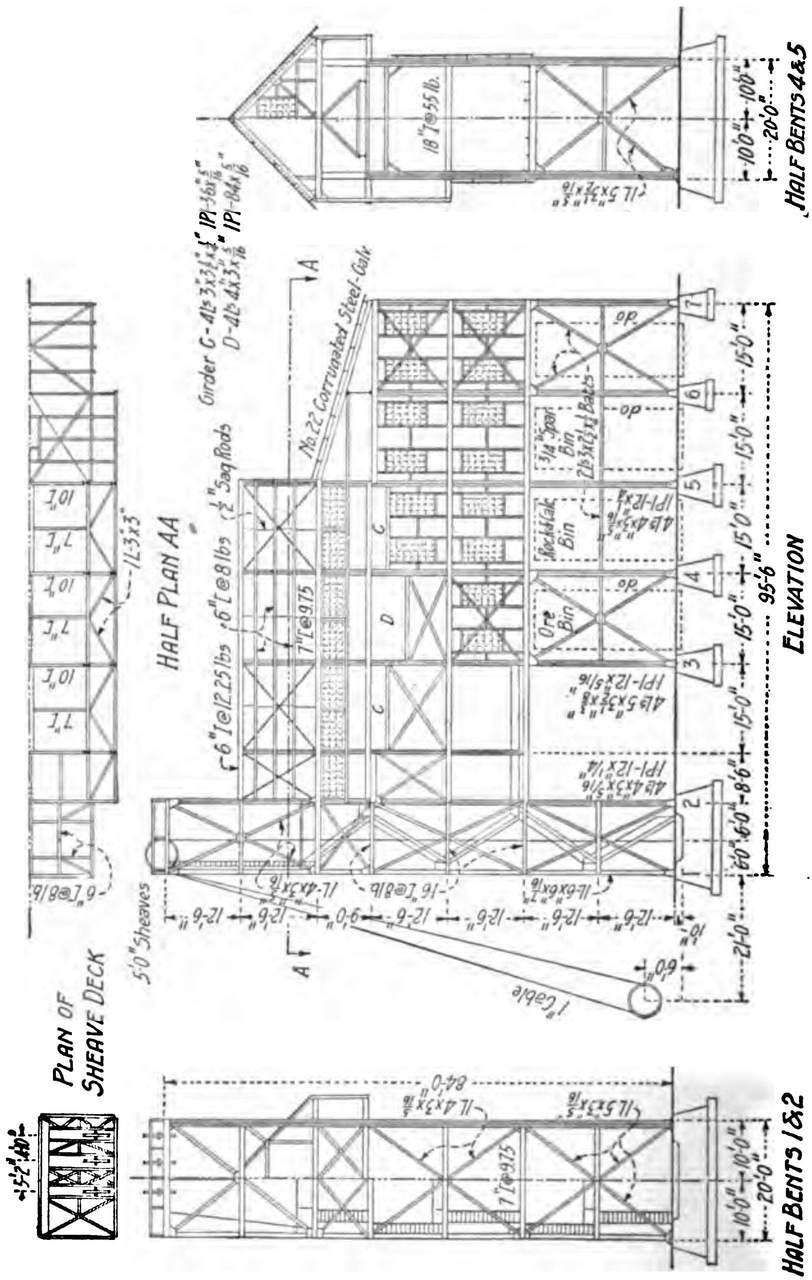

FIG. 219 b. SIZING AND SORTING BUILDING, ROSICLARE LEAD AND FLUORSPAR MINES.

fluorspar is hoisted in steel mine cars having a capacity of one ton. The hoist has a capacity of 50 tons per hour, or 500 tons in 10 hours.

The plant consists of three buildings: (1) the sizing and sorting building, Fig. 219 b, 20 ft. wide by 90 ft. long and 71 ft. 6 in. high; (2) the concentrator building, 36 ft. wide by 90 ft. long, with 30 ft. side posts; and (3) the power house, 36 ft. wide by 90 ft. long, with 30 ft. side posts. The buildings were designed for the following loads: Working load on 1 in. hoisting rope, 5 tons; roof load, 40 lbs. per sq. ft.; wind load, 30 lbs. per sq. ft.; floor loads, 200 lbs. per sq. ft. The allowable unit stresses were: tension, 16,000 lbs.; compression, 16,000 lbs. reduced by a standard column formula; for wind load stresses the above allowable stresses were increased 25 per cent. The roof covering for all three buildings is No. 22 gage galvanized corrugated steel, carried on 6″ @ 8 lb. channel purlins. The sizing and sorting building is covered with No. 22 gage galvanized corrugated steel down to the picking belt floor. The concentrator building and the power house have brick walls.

Operation of Plant.—The flow sheet shown in Fig. 219 e gives the operation of the plant in detail. From the cage the mine cars are run on tracks and are dumped on the grizzlies. The grizzlies are made of bars 1½ in. high and ¾ in. thick at the top and ½ in. thick at the bottom, spaced 2½ in. centers. From the oversize No. 1 lump fluorspar is picked and thrown on the conveyors, and the remainder of the oversize goes through a crusher. The material passing the grizzly and the crusher passes into two 150 ton steel bins on the floor below. The fluorspar is fed from the bins into two shaking screens, each 2 ft. 9 in. wide by 17 ft. long with ¾ in. mesh. The oversize is delivered to a rotary drier 36 in. in diameter and 25 ft. long. From the drier the product may be barreled for shipment or may be run through a Griffin mill, which grinds the spar so that it will pass a No. 30 mesh screen, and then into four storage bins.

Ore Separation.—From the shaking screens the undersize is carried by water through a 9 in. pipe to the jig house, while the oversize from the screens falls on a picking belt, and the lead and zinc ore are sorted out by 9 to 15 men. The fine material and the overrun of the fluorspar is crushed and is run through jigs, which separate the fluorspar and the lead and zinc ore.

The weight of the structural steel and corrugated steel in the three buildings is as follows: (1) sizing and sorting building, structural steel

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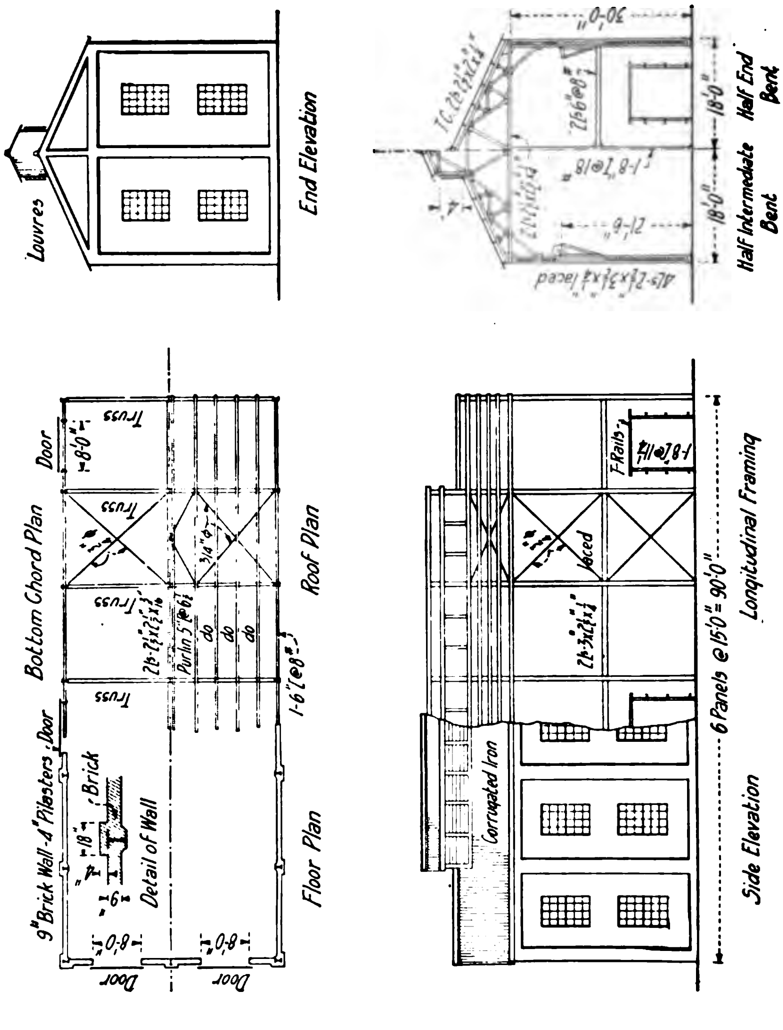

FIG. 219 c. CONCENTRATOR BUILDING, ROSICLARE LEAD AND FLUORSPAR MINES.

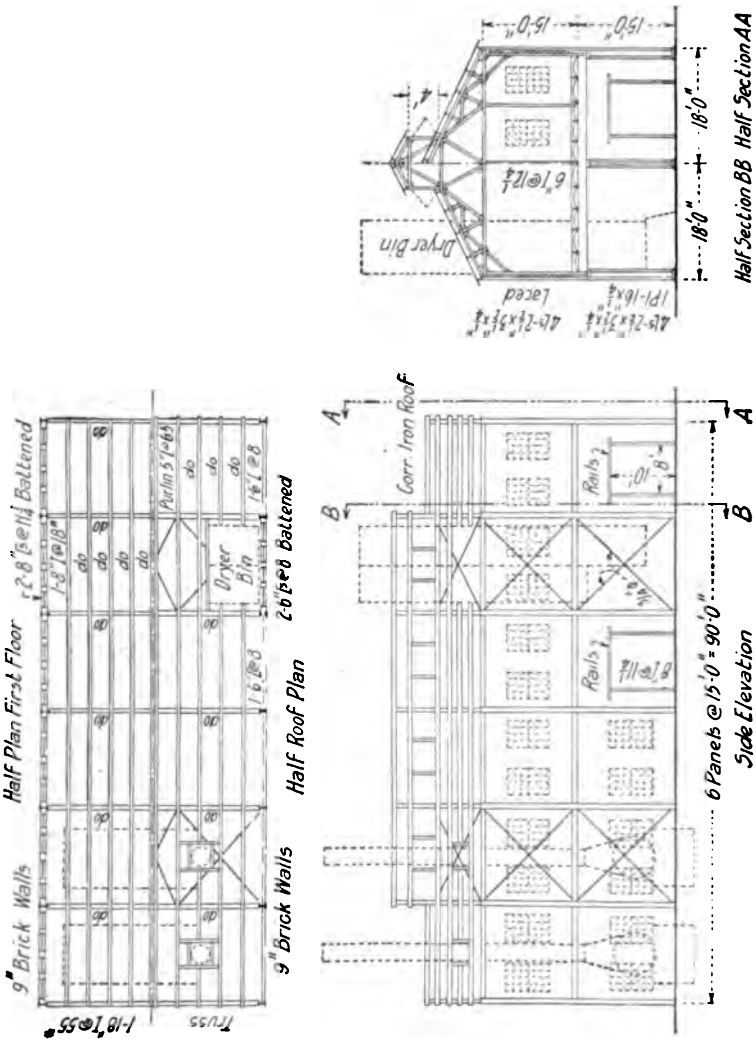

FIG. 219 d. POWER HOUSE, ROSICLARE LEAD AND FLUORSPAR MINES.

250,000 lbs., corrugated steel 15,500 lbs.; (2) concentrator building, structural steel 28,000 lbs., corrugated steel 6,300 lbs.; (3) power plant, structural steel 78,000 lbs., corrugated steel 6,300 lbs.

Timber Trestles.—In a framed timber trestle the ties which carry the track are carried on timber stringers, varying from 4 in number for light loads to 6 for heavy loads. The stringers are carried by

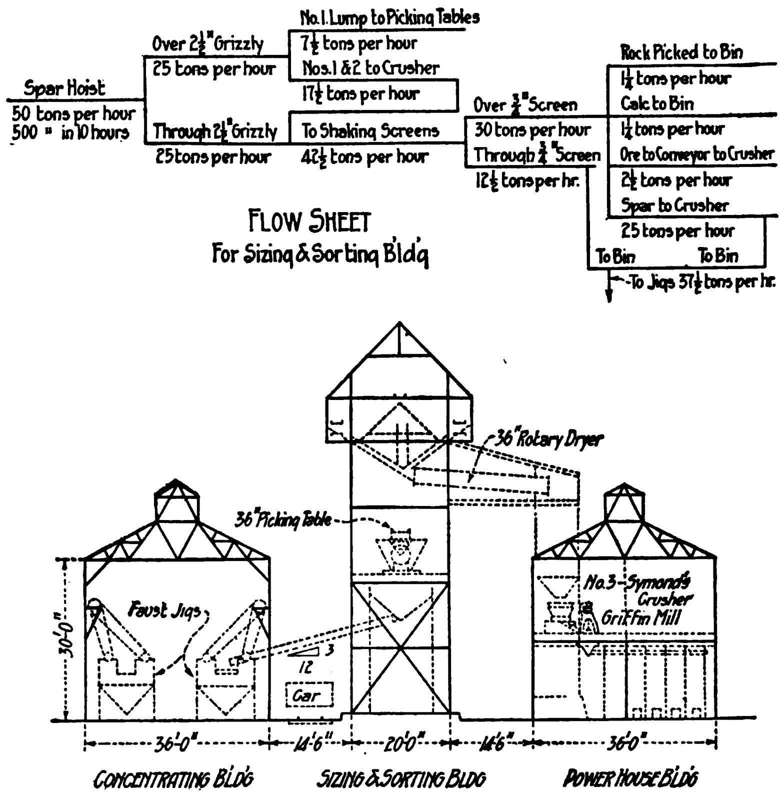

FIG. 219 e.

trestle bents, each of which consist of a cap on which the stringers rest, posts which carry the cap, and a sill upon which the posts rest. For a single story bent the sill rests either on piles, blocks of timber called mudsills, or on masonry piers. The bent is braced transversely and longitudinally by means of horizontal and diagonal bracing. Where the bents are more than about 30 ft. high two trestle bents are braced

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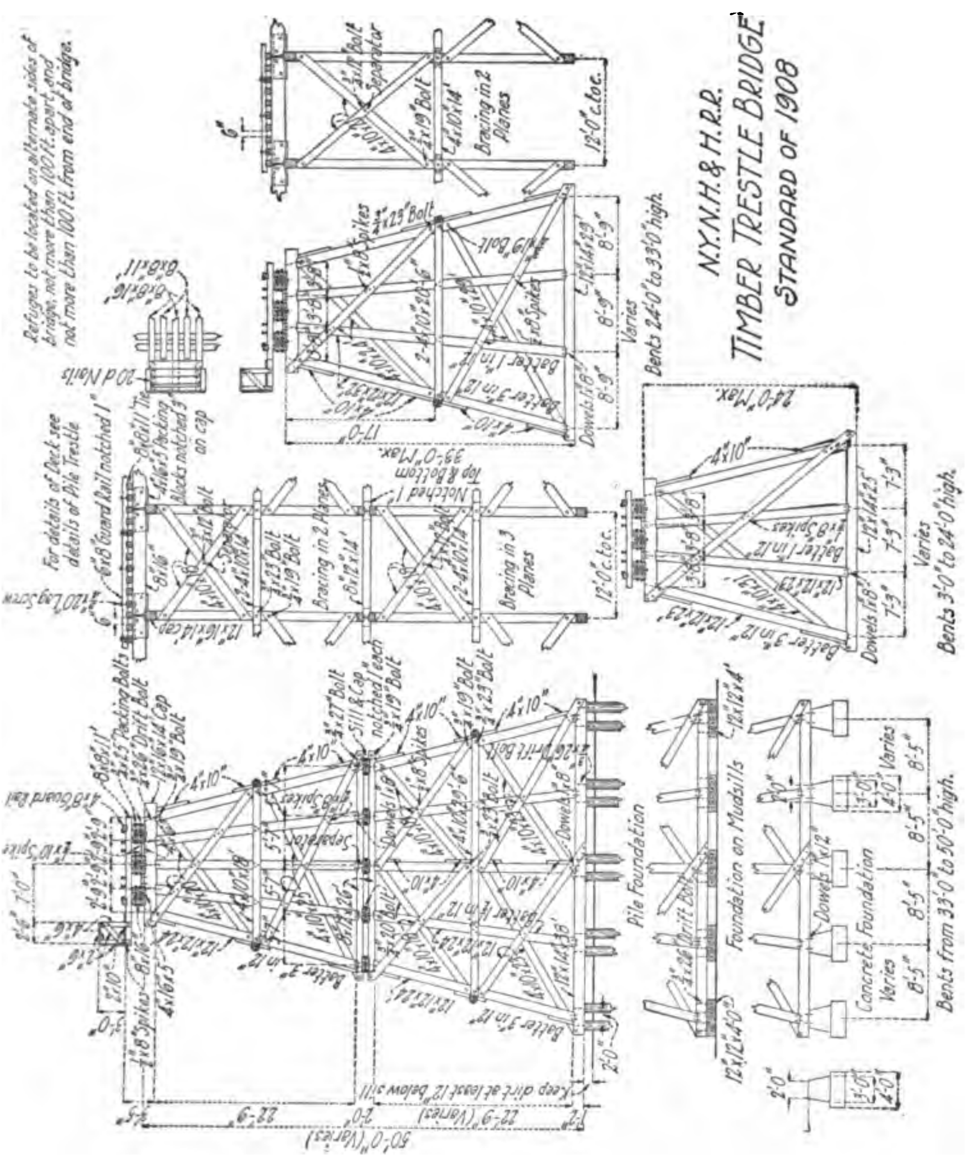


FIG. 219 f.

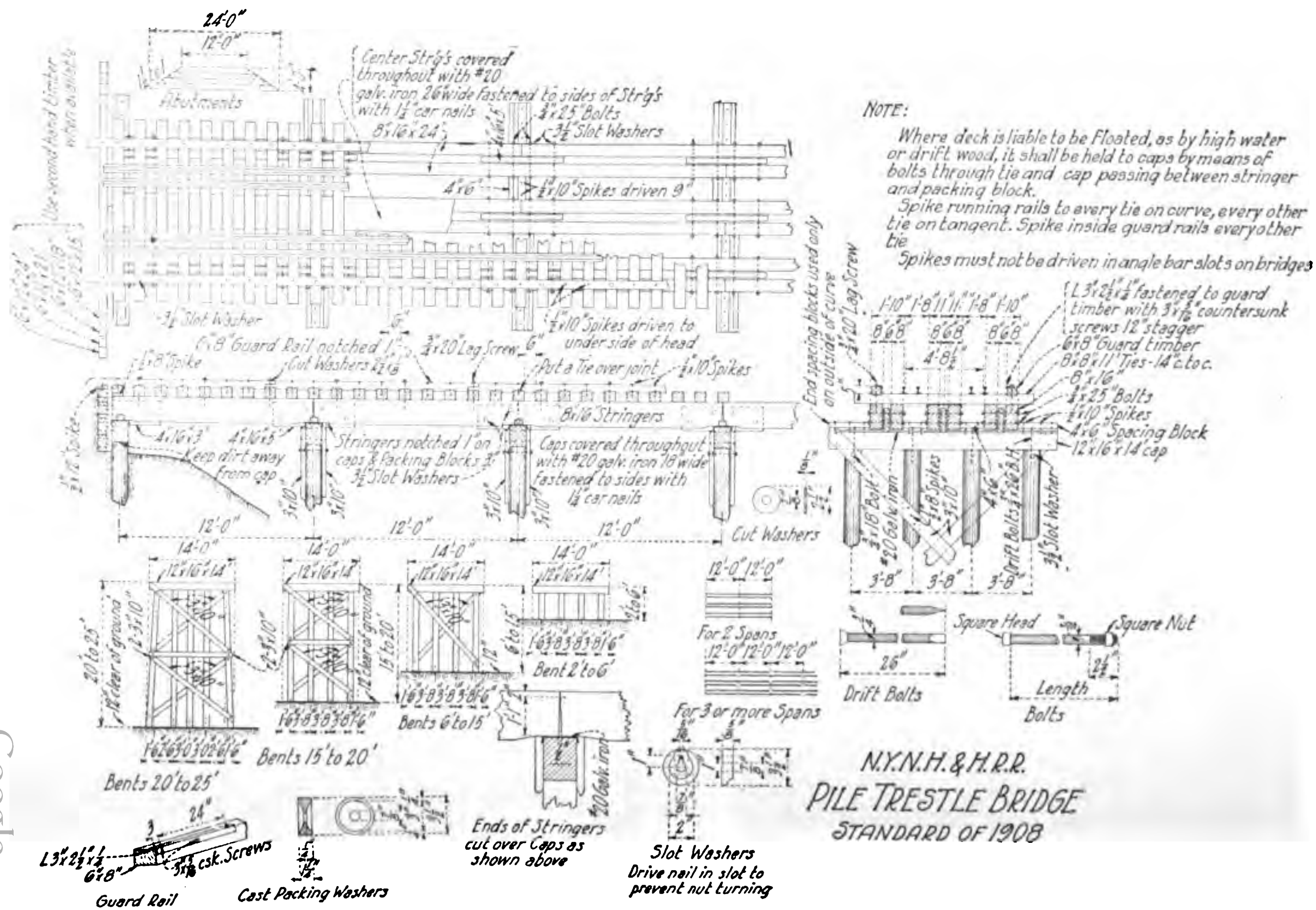

NOTE:
Where deck is liable to be Floated, as by high water or drift wood, it shall be held to caps by means of bolts through tie and cap passing between stringer and packing block.
Spike running rails to every tie on curve, every other tie on tangent. Spike inside guard rails every other tie
Spikes must not be driven in angle bar slots on bridges
24'-0"
12'-0"
Abutments
Center Strg's covered throughout with #20 galv. iron 26" wide fastened to sides of Strg's with 1¼" car nails
8"x16"x24"
¾"x25" Bolts
3½" Slot Washers
4"x6"
⅜"x10" Spikes driven 9"
3½" Slot Washer
6"x8" Guard Rail notched 1"
Cut Washers
¾"x20" Leg Screw
⅜"x10" Spikes driven to under side of head
Put a Tie over joint
⅜"x10" Spikes
8x16 Stringers
Stringers notched 1" on caps & Packing Blocks 3"
3½" Slot Washers
Caps covered throughout with #20 galv. iron 18" wide fastened to sides with 1¼" car nails
Keep dirt away from cap
12'-0"
12'-0"
12'-0"
Cut Washers
End spacing blocks used only on outside of curve
¾"x20" Leg Screw
1'-10" 1'-8" 11" 11" 1'-8" 1'-10"
8"6"8" 8"6"8" 8"6"8"
4'-8½"
L 3"x2½"x¼" fastened to guard timber with ⅜"x1⅝" countersunk screws 12" stagger
6"x8" Guard timber
8"x8"x11' Ties-14" c. to c.
8"x16"
¾"x25" Bolts
⅜"x10" Spikes
4"x6" Spacing Block
12"x16"x14' cap
#20 Galv. iron
Drift Bolts
3½" Slot Washer
3'-8" 3'-8" 3'-8"
14'-0"
12"x16"x14'
20' to 25'
12' clear of ground
15' to 20'
6' to 15'
Bent 2' to 6'
Bents 6' to 15'
Bents 15' to 20'
Bents 20' to 25'
12'-0" 12'-0"
For 2 Spans
12'-0" 12'-0" 12'-0"
For 3 or more Spans
26"
Drift Bolts
Square Head
Square Nut
Length
Bolts
#20 Galv. iron
Ends of Stringers cut over Caps as shown above
Slot Washers
Drive nail in slot to prevent nut turning
L 3"x2½"x¼"
6"x8"
csk. Screws
Guard Rail
Cast Packing Washers
N.Y.N.H.&H.R.R.
PILE TRESTLE BRIDGE
STANDARD OF 1908

FIG. 219 g.

together to form a tower, and the posts are made of two or more sections. The caps are commonly bolted to the posts with drift bolts, square or round rods ¾ to 1 in. diameter and 15 to 24 in. long, driven in holes bored somewhat smaller in diameter than the drift bolt. The posts are fastened to the sills by means of dowels, rods or bars similar to drift bolts, except that the holes bored to receive them must be larger and deeper, so that the post may be forced over the dowel which has been driven in the sill. The bracing is fastened at the main joints by means of bolts and at secondary joints by means of spikes. The detail design of the standard framed timber trestle of the N. Y., N. H. & H. R. R. is shown in Fig. 219 f.

Pile Trestles.—In a pile trestle the trestle bent which carries the stringers is made of four or more piles, which have been driven to a good refusal and sawed off at the proper height to receive the cap. The pile bent is cross-braced to give greater rigidity. The detail design of the standard pile trestle of the N. Y., N. H. & H. R. R. is shown in Fig. 219 g.

Retaining Walls.—A plain gravity concrete retaining wall and a reinforced concrete retaining wall designed by the C. B. & Q. R. R. for the same conditions are shown in Fig. 219 h. A reinforced concrete wall of the slab type is shown in Fig. 219 i. This wall was designed by

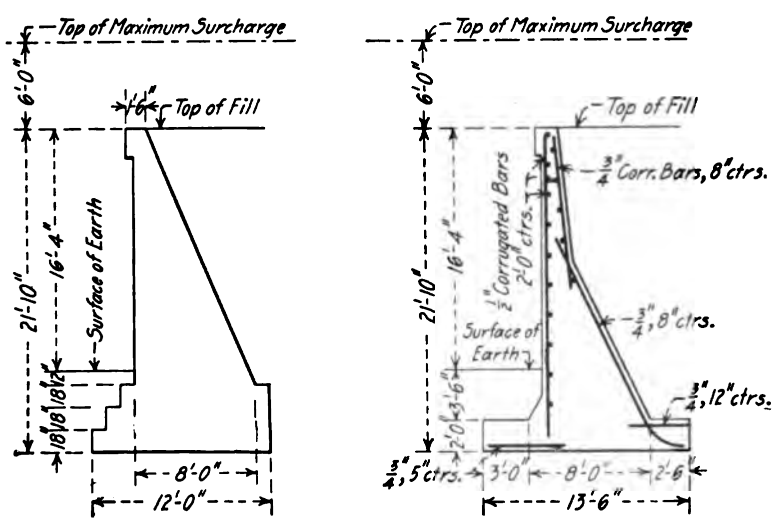
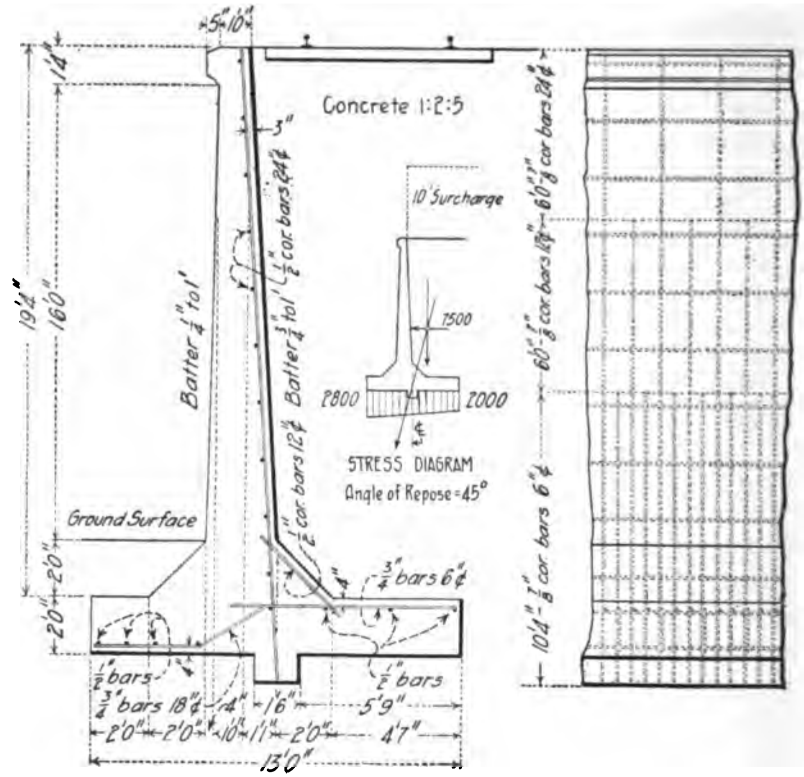


FIG. 219 h. C. B. & Q. R. R. RETAINING WALLS.



SECTION FRONT ELEVATION

FIG. 219 i. SLAB WALL, ILLINOIS CENTRAL R. R.

the Illinois Central R. R. for the following conditions: weight of concrete, 150 lbs. per cu. ft.; weight of filling, 100 lbs. per cu. ft.; angle of repose, $\phi = 45°$; concrete 1:2:5. For additional examples of retaining walls see the author's "The Design of Walls, Bins and Grain Elevators."

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PART III.

DETAILS OF DESIGN AND COST OF MINE STRUCTURES.

Introduction.—This part of the book includes a discussion of the details of the design of mine structures, together with tables, drawings and data to be used in design; also a discussion of the costs of mine structures in detail. Only such tables as are not easily available in the "Carnegie" and "Cambria" hand-books are given. The discussion on general design is supplementary to the discussion in Parts I and II. In the discussion of costs of mine structures emphasis has been placed on the effect of costs on the design of the structure rather than to give data to be blindly followed.

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CHAPTER XIV.

DETAILS OF THE DESIGN OF STEEL STRUCTURES.

Stresses in Crane Girders.—It is proved in the author's "The Design of Steel Mill Buildings," Chapter IX, "that where a beam or girder carries several moving concentrated or wheel loads, the maximum bending moment will come under a heavy wheel near the center of the moving loads, when the wheel is as far from one end of the beam or girder as the center of gravity of all the loads then on the beam is from the other end." Where there are three equal loads equally spaced the maximum bending moment will come at the center of the span when the middle wheel is over that point.

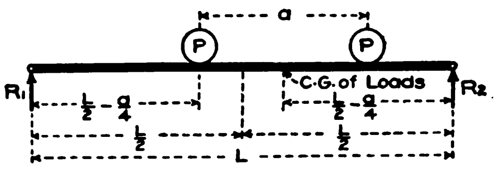

FIG. 220.

For two equal loads $P=P$ at a fixed distance, a, apart as in the case of a traveling crane, Fig. 220, the maximum moment will occur under one of the loads when the load is at a distance from the end of the beam, $x=L/2-a/4$.

Taking moments about the right reaction we have

$$R_1=P(L-a/2)/L \tag{127}$$

and the maximum bending moment is

$$M=R_1(L/2-a/4)=P(L-a/2)^2/2L \tag{128}$$

There will be a maximum moment when either of the loads satisfies the above criterion, the bending moments being equal.

By equating the maximum moment above to the moment due to a single load at the center of the beam, it will be found that the above criterion holds only when

$$a<0.586\,L$$

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Where two unequal moving loads are at a fixed distance apart the greater maximum bending moment will always come under the heavier load.

The maximum end shear at the left support for a system of concentrated loads on a simple beam will occur when the left reaction, R_1, is a maximum. This will occur when one of the wheels is infinitely near the left abutment (usually said to be over the left abutment). The load which produces maximum end shear can be easily found by trial.

The maximum shear at any point in the beam will occur when one of the loads is over the point, the maximum shear in crane girders being easily found by trial.

DESIGN OF PLATE GIRDERS.—The maximum moments and shears are found as just described. If the plate girder were designed by means of its moment of inertia, as in the case of rolled sections, about $\frac{1}{6}$ of the web would be effective as flange area to take the bending moment; or deducting rivets about $\frac{1}{8}$ would be found effective. It is, however, the common practice to assume that all the moment is taken by the flanges, and that all the shear is taken by the web, and this assumption will be made in the discussion which follows.

Let F = area of one flange, not including the included web plate;
h = height of the girder between centers of gravity of flanges;
t = thickness of web plate in in.;
f = allowable stress in flanges in lbs. per sq. in.;
$A = t \cdot h$ = area of web plate in sq. in.

Flange Stress.—The stress, $F \cdot f$, in the flanges at any point in a plate girder is

$$f \cdot F = M/h \tag{129}$$

where M = bending moment in in.-lbs., and h = distance in in. between centers of gravity of flange areas (effective depth), (a) Fig. 221. The tension flanges of plate girders are designed as above, and the compression flanges are made with the same gross area.

If one eighth of the web is taken as effective as flange area the net flange area will be equal to area F minus one eighth of the area of the web plate $= F - t \cdot h/8$.

Web.—The web plate should not be less than $\frac{3}{8}$ in. in thickness, although $\frac{5}{16}$ in. plates may be used if provided with sufficient stiffeners.

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The shear in the web is commonly assumed as uniformly distributed over the entire cross-section of the plate.

Stiffeners.—There is no rational method for the design of stiffeners. If they are placed at distances apart not exceeding the depth

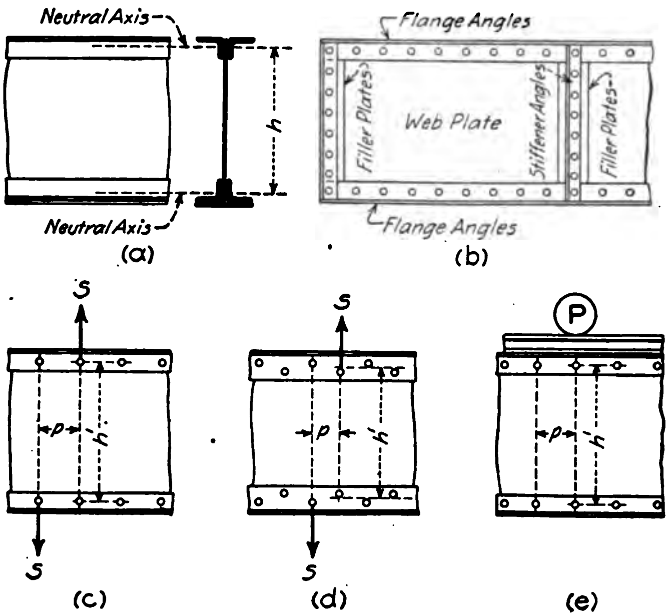

FIG. 221.

of the girder, nor more than 5 ft., where the shearing stress is greater than given by the formula—allowed shearing stress $= 12{,}500 - 90H$, where $H =$ ratio of depth to thickness of web plate, the stiffeners will be near enough together. Where the shearing stress is less than given by the above formula, stiffeners may be omitted or spaced as desired.

Stiffeners are commonly designed as columns, free to move in a direction at right angles to the web, with an allowed stress $P = 12{,}000 - 55 \times l/r$, where $l =$ one half the depth of the girder and $r =$ radius of gyration of the stiffener angles about the center of the web, both in inches. Stiffeners should be provided at all points of support and under all concentrated loads, and should contain enough rivets to transfer the vertical shear.

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Web Splice.—In the plain web splice shown in Fig. 222, the rivets take a uniform shear equal to S/n, where S = total shear and n = number of rivets on one side of the splice, and a shear due to the shearing stress not being applied at the center of gravity of the rivets. This is the problem of the eccentric riveted connection, which has been discussed in "The Design of Steel Mill Buildings," Chapter XV.

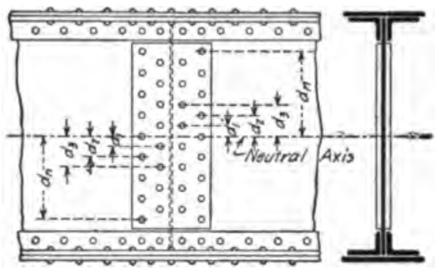

FIG. 222.

If the web is assumed to take part of the bending moment there will be an additional shear due to bending moment.

Rivets in the Flanges.—In Fig. 221, let S = the shear in the girder at the given section, h' = distance between rivet lines, p = the pitch of

TABLE XXVIII.

TYPICAL HAND CRANES.

Capacity in Tons.	Span, Ft.	Wheel Base.	Maximum Wheel Load, Lbs.	Vertical Clearance.	Side Clearance.	Weight of Rails, Lbs. per Yard for I Beams.	Plate Girders.
2	30	4'-0"	3,100	4'-0"	7"	30	30
	50	5'-0"	4,000	4'-0"	7"	30	30
4	30	4'-0"	5,400	4'-6"	8"	30	30
	50	5'-0"	6,500	4'-6"	8"	30	30
6	30	6'-0"	8,000	5'-0"	9"	30	35
	50	7'-0"	9,200	5'-0"	9"	30	35
8	30	6'-0"	10,500	5'-0"	10"	35	40
	50	7'-0"	11,800	5'-0"	10"	35	40
10	30	7'-0"	13,000	5'-0"	10"	40	40
	50	8'-0"	14,400	5'-0"	10"	40	40
16	30	7'-0"	20,700	5'-6"	10"	45	45
	50	8'-0"	22,300	5'-6"	10"	45	45
20	30	7'-0"	26,000	5'-6"	10"	50	50
	50	8'-0"	28,000	5'-6"	10"	50	50
25	30	7'-0"	32,300	6'-0"	12"	50	55
	50	8'-0"	35,000	6'-0"	12"	50	55

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TABLE XXIX.

TYPICAL ELECTRIC TRAVELING CRANES.

Capacity, Tons.	Span, Ft.	Wheel Base.	Maximum Wheel Load, Lbs.	Total Weight of Crane, Lbs.	Side Clearance.	Vertical Clearance.	Weight of Rail, Lbs. per Yard, for	
							Plate Girders.	I Beams.
$3\frac{1}{2}$	to 30	6′ 9″	9,600	16,700	$9\frac{1}{2}$″	4′ 10″	40	40
$3\frac{1}{2}$	40	6′ 11″	10,400	19,200	$9\frac{1}{2}$″	4′ 11″	40	40
$3\frac{1}{2}$	60	10′ 0″	12,600	27,700	10″	5′ 3″	40	40
5	to 30	8′ 0″	11,600	19,500	10″	6′ 0″	40	40
5	40	8′ 6″	12,800	22,400	10″	6′ 0″	40	40
5	60	10′ 0″	15,500	31,300	10″	6′ 0″	40	40
$7\frac{1}{2}$	to 30	8′ 6″	14,900	22,300	10″	6′ 0″	40	40
$7\frac{1}{2}$	40	8′ 8″	16,200	24,900	10″	6′ 0″	40	40
$7\frac{1}{2}$	60	10′ 0″	19,100	34,100	10″	6′ 0″	40	40
10	to 30	8′ 6″	18,500	23,500	10″	6′ 0″	45	40
10	40	8′ 8″	19,800	28,400	10″	6′ 0″	45	40
10	60	10′ 0″	22,700	37,800	10″	6′ 0″	45	40
15	to 30	9′ 6″	25,700	29,600	12″	6′ 7″	50	50
15	40	9′ 6″	27,100	33,900	12″	6′ 9″	50	50
15	60	10′ 0″	29,900	44,000	12″	6′ 11″	50	50
20	to 30	9′ 6″	32,300	34,200	12″	7′ 1″	55	50
20	40	9′ 6″	34,300	38,800	$12\frac{1}{2}$″	7′ 3″	55	50
20	60	10′ 0″	38,300	50,700	13″	7′ 6″	55	50
25	to 40	9′ 0″	40,200	44,500	13″	7′ 7″	60	50
25	60	10′ 0″	45,300	59,500	13″	8′ 0″	60	50
30	to 40	9′ 8″	46,200	51,100	13″	8′ 0″	70	60
30	60	10′ 4″	52,200	68,000	14″	8′ 6″	70	60
40	to 40	10′ 8″	61,000	69,300	16″	8′ 9″	80	60
40	60	11′ 2″	68,600	87,000	16″	9′ 1″	80	60
50	to 40	11′ 2″	74,000	77,100	16″	9′ 5″	100	60
50	60	11′ 6″	86,000	98,500	16″	9′ 5″	100	60

the rivets, and r = the resistance of one rivet (r is usually the safe bearing on the rivet in the web). Then taking moments about the lower right hand rivet, we have

$$S \cdot p = r \cdot h', \quad \text{and} \quad p = r \cdot h'/S \tag{130}$$

Where the rivets are in double rows as shown in (d), the distance h' is taken as a mean of the distances for the two lines.

The crane loads produce an additional shear in the rivets, (e) Fig. 221, which will now be investigated. We will assume that the rail distributes the load over a distance of 25 in.; this distance will be less for

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light rails and more for heavy rails. The maximum vertical shear on one rivet will be $P \cdot p \div 25 = 0.04 P \cdot p$. The horizontal stress due to bending moment is $r = S \cdot p \div h'$, and the resultant stress from the two sources will be

$$r' = \sqrt{(0.04 P \cdot p)^2 + \left(\frac{S \cdot p}{h'}\right)^2}$$

and solving for p

$$p = \frac{r'}{\sqrt{(0.04P)^2 + \left(\frac{S}{h'}\right)^2}} \qquad (131)$$

WEIGHT OF CRANES.—The weights and other data for hand cranes are given in Table XXVIII, and similar data are given for electric traveling cranes in Table XXIX.

FLAT PLATES.—The analysis of the stresses in flat plates supported or fixed at their edges is extremely difficult. The following formulas by Grashof may be used: The coefficient of lateral contraction is taken as $\frac{1}{4}$. For a full discussion of these formulas based on Grashof's "Theorie Der Elasticitat und Festigkeit" see Lanza's Applied Mechanics.

1. *Circular plate of radius r and thickness t, supported around its perimeter and loaded with w per square inch.*—Let f = maximum fiber stress, v = maximum deflection, and E = modulus of elasticity,

$$f = \frac{117}{128} \frac{w \cdot r^2}{t^2} \qquad (132)$$

$$v = \frac{189}{256} \frac{w \cdot r^4}{E \cdot t^3} \qquad (133)$$

2. *Circular plate built in or fixed at the perimeter.*

$$f = \frac{45}{64} \frac{w \cdot r^2}{t^2} \qquad (134)$$

$$v = \frac{45}{256} \frac{w \cdot r^4}{E \cdot t^3} \qquad (135)$$

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3. *Rectangular plate of length a, breadth b, and thickness t, built in or fixed at the edges and carrying a uniform load w per square inch.*— Let f_a be the unit stress parallel to a, f_b be the unit stress parallel to b, and $a > b$.

$$f_a = \frac{b^4 \cdot w \cdot a^2}{2(a^4 + b^4)t^2}; \quad f_b = \frac{a^4 \cdot w \cdot b^2}{2(a^4 + b^4)t^2} \tag{136}$$

$$v = \frac{a^4 \cdot b^4 \cdot w}{(a^4 + b^4)32E \cdot t^3} \tag{137}$$

For a square plate $a = b$,

$$f = \frac{w \cdot a^2}{4t^2} \tag{138}$$

$$v = \frac{w \cdot a^4}{64E \cdot t^3} \tag{139}$$

The strength of plates simply supported on the edges is about ⅔ the strength of plates fixed. Plates riveted or bolted around the edges may be considered as fixed.

Diagram for Square Plates.—The safe loads on square plates for a fiber stress of 10,000 pounds per square inch may be obtained from the diagram in Fig. 224. As an example, required the safe load for a ¼ in. plate 3 feet square. Begin at 3 on the bottom of the diagram, follow upward to the line marked ¼ in. plate, from the intersection follow to the left edge and find 280 lbs. per sq. ft. For any other fiber stress multiply the safe load found from the diagram by the ratio of the fiber stresses. To use the diagram for a rectangular plate take a square plate having the same area.

Buckle Plates.—Buckle plates are made by "dishing" flat plates as in Table XXX. The width of the buckle W, or length L, varies from 2 ft. 6 in. to 5 ft. 6 in. The buckles may be turned with the greater dimension in either direction of the plate. Several buckles may be put in one plate, all of which must be the same size and symmetrically placed. Buckle plates are made ¼ in., 5/16 in., ⅜ in. and 7/16 in. in thickness. The common standard sizes are given in Table XXX.

Buckle plates should be firmly bolted or riveted around the edges with a maximum spacing of 6 inches, and should be supported transversely between the buckles. The process of buckling distorts the

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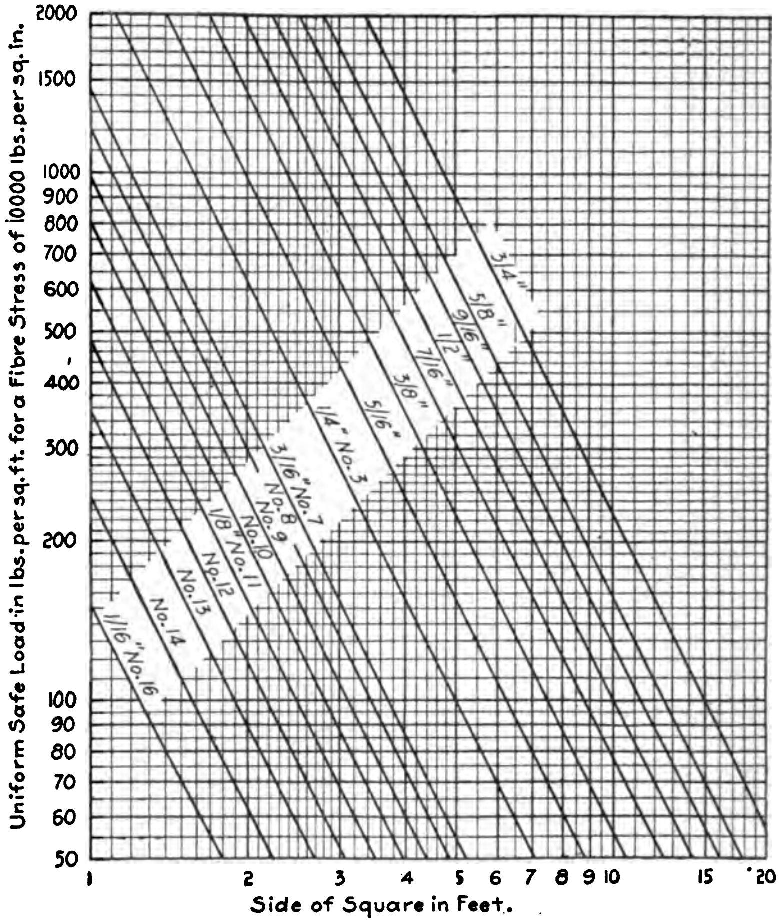

FIG. 224. SAFE UNIFORM LOAD PER SQUARE FOOT ON FLAT PLATES FOR A FIBER STRESS OF 10,000 LBS. PER SQUARE INCH.

plate and an extra width should be ordered and the plate should be trimmed after the process is complete.

Strength of Buckle Plates.—The safe load for a buckle plate with buckles placed up, is approximately given by the formula

$$W = 4f \cdot R \cdot t \tag{140}$$

where W = total safe uniform load in lbs.;

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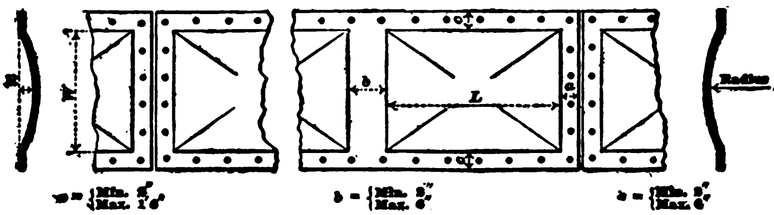
f = safe unit stress in pounds per square inch;
R = depth of buckle in inches;
t = thickness of plate in inches.

Where buckle plates are riveted and the buckle placed down, the safe load is from 3 to 4 times that given above.

TABLE XXX.

AMERICAN BRIDGE CO.'S STANDARD BUCKLE PLATES.



No. of Plate.	Size of Buckle in Feet and Inches. Length. L	Width. W	Rise in Inches. R	Rad. of Buckle in Feet and Inches. Length. L	Width. W	Maximum No. of Buckles.	No. of Plate.	Size of Buckle in Feet and Inches. Length. L	Width. W	Rise in Inches. R	Rad. of Buckle in Feet and Inches. Length. L	Width. W	Maximum No. of Buckles.
1	3-11	4-6	3½	6-8	8-9	7	20	2-9	2-6	2½	4-7	3-10	10
2	4-6	3-11	3½	8-9	6-8	6	21	2-6	2-6	2½	3-10	3-10	10
3	3-11	3-6	3	7-9	6-3	7	22	3-5	3-6	3	5-11	6-3	8
4	3-6	3-11	3	6-3	7-9	8	23	3-6	3-5	3	6-3	5-11	8
5	3-9	3-9	3	7-1	7-1	8	24	3-6	3-9	3	6-3	7-1	8
6	3-1	3-9	3	4-10	7-1	9	25	3-9	3-6	3	7-1	6-3	8
7	3-9	3-1	3	7-1	4-10	8	26	3-1	3-2	3	4-10	5-1	9
8	3-8	3-8	2	10-2	10-2	8	27	3-2	3-1	3	5-1	4-10	9
9	2-8	3-8	2	5-5	10-2	10	28	3-1	3-0	3	4-10	4-7	9
10	3-8	2-8	2	10-2	5-5	8	29	3-0	3-1	3	4-7	4-10	9
11	2-2	3-8	2	3-7	10-2	10	30	2-0	2-6	2½	2-6	3-10	10
12	3-8	2-2	2	10-2	3-7	8	31	2-6	2-0	2½	3-10	2-6	15
13	3-0	3-0	2	6-10	6-10	9	32	3-6	5-6	3½	5-4	13-1	5
14	2-9	2-9	3	3-10	3-10	10	33	5-6	3-6	3½	13-1	5-4	1
19	2-6	2-9	2½	3-10	4-7	10	34	4-0	4-0	3	8-1	8-1	7

Plates are made ¼″, $\frac{5}{16}$″, ⅜″ or $\frac{7}{16}$″ thick.
Buckles of different sizes should not be used in the same plate.
Rivets generally ⅝″ or ¾″ diameter.

STRUCTURAL DRAWINGS.—In making shop or working drawings for structural steel work the members may be detailed in the position that they are to occupy, as in the shop drawings for the roof truss in Fig. 227, or the members may be detailed separately, an erection or general plan being used for assembling. The first method is commonly used for steel frame buildings, while the second method is

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commonly used for head frames and complicated structures. Where the members are detailed in place the outline of the structure is laid off to a scale of from $\frac{1}{8}$ in. to $\frac{1}{2}$ in. to 1 ft., while the details of the members are drawn to a scale of from $\frac{3}{4}$ in. to $1\frac{1}{2}$ in. to 1 ft. The figured dimensions always control in structural details, no attempt being made to draw members to exact scale.

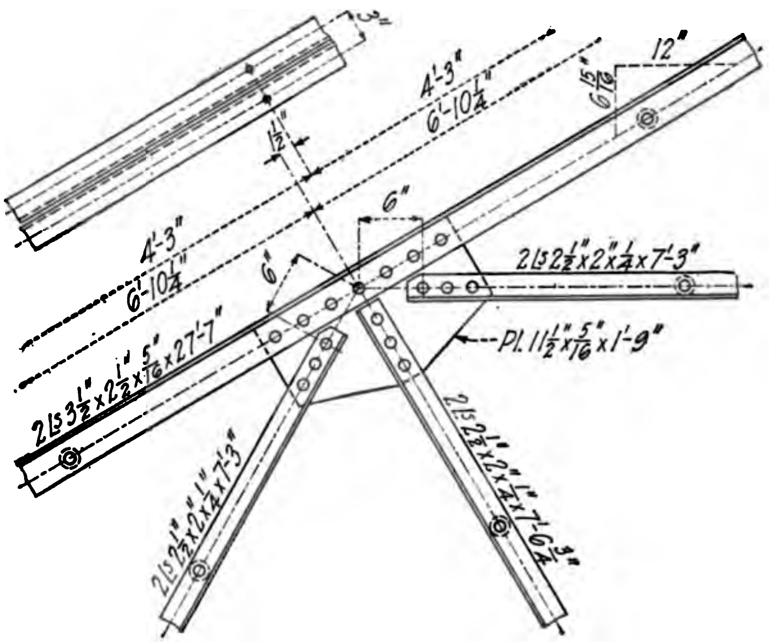

FIG. 225. JOINT OF ROOF TRUSS, "SKETCH" DETAILED.

The shop drawings required by a bridge or structural shop will depend largely upon the amount of work that is done in the templet shop. Where the truss or framework is laid out in detail on the floor of the templet shop, only the main dimensions, sizes of members, lengths of members and number and arrangement of the rivets in the connections need be given. Where the framework is not laid out in detail on the floor of the templet shop, or where it is desired that the rivets be located exactly as determined by the engineer and a record preserved, the exact rivet spacing is required in addition to the data required in the first method. The joint of a roof truss detailed according to the first method is shown in Fig. 225, while the same joint is com-

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pletely detailed in Fig. 226. The shop drawings for a steel roof truss are shown in Fig. 227. This roof truss is completely detailed. In detailing, the figures are commonly written above the dimension lines, as shown in Fig. 225 and Fig. 226, and not as shown in Fig. 227.

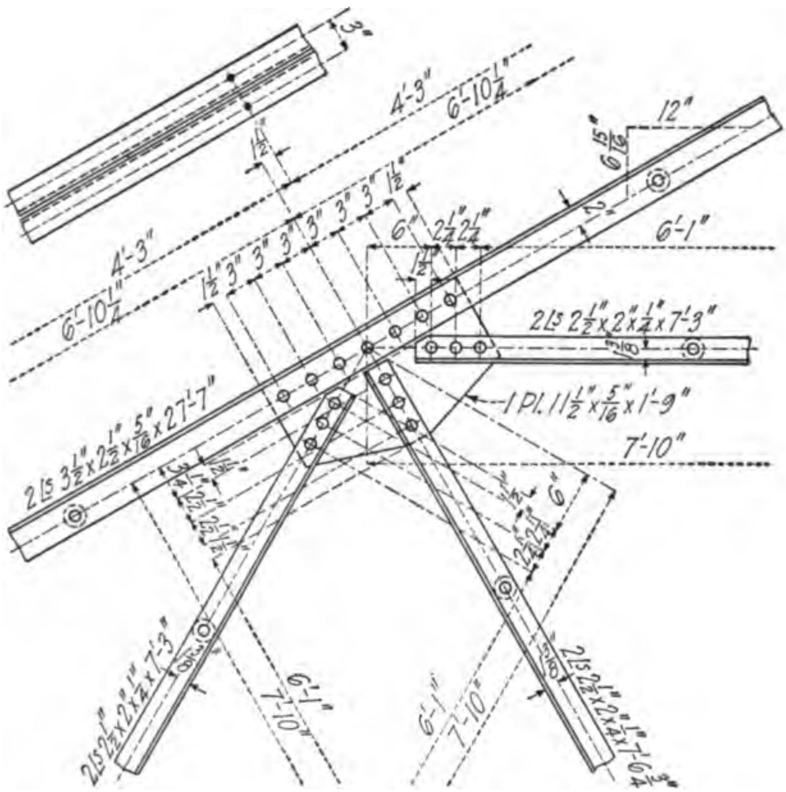

Fig. 226. Joint of Roof Truss, Completely Detailed.

Lacing Bars.—The American Bridge Company's standard details for lacing bars are given in Fig. 228. Where laced columns are required to carry heavy loads or take transverse flexure, the sizes of the lacing bars should be calculated, or a solid web should be used. For the design of lacing bars see the author's "The Design of Highway Bridges," Chapter XIII.

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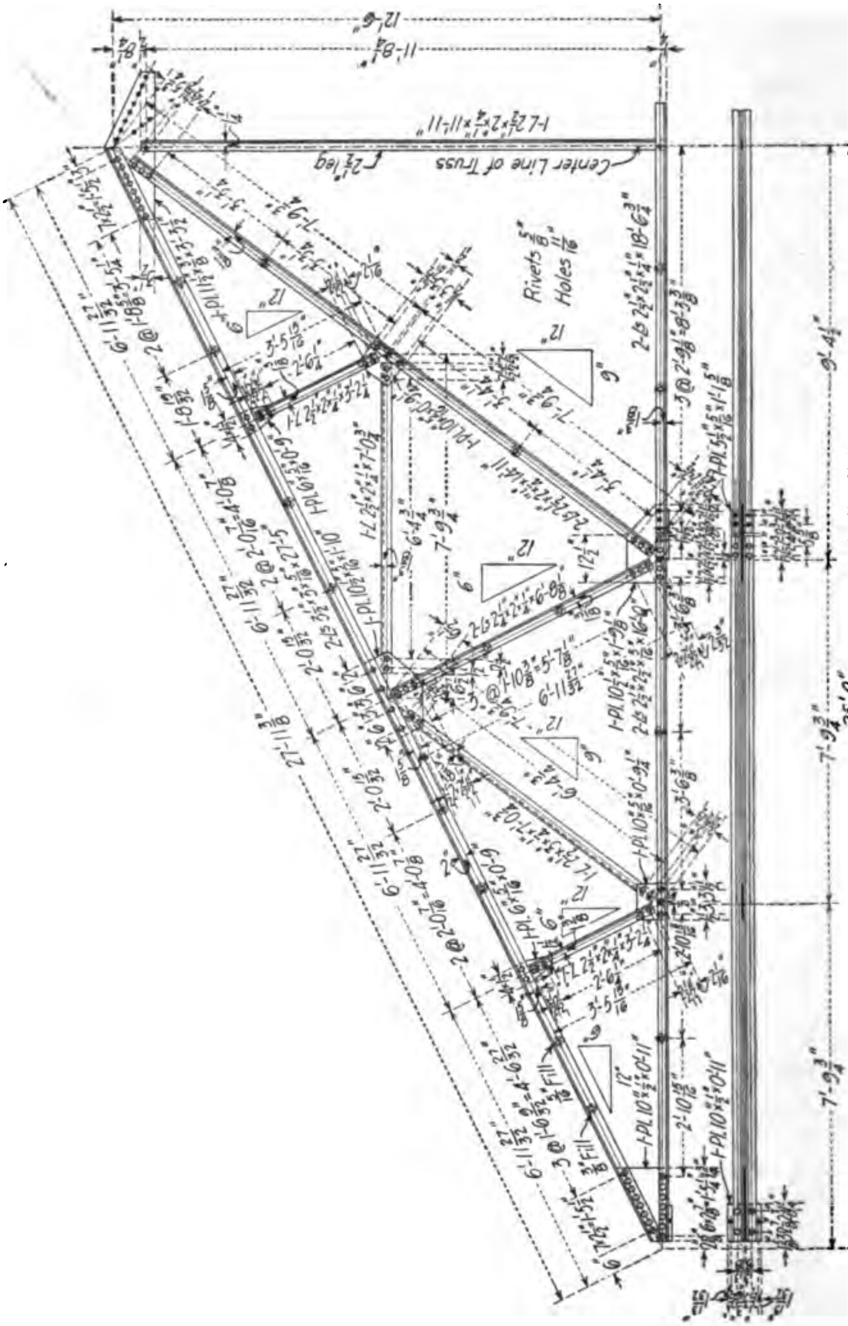

FIG. 227. DETAIL SHOP PLANS FOR A ROOF TRUSS.

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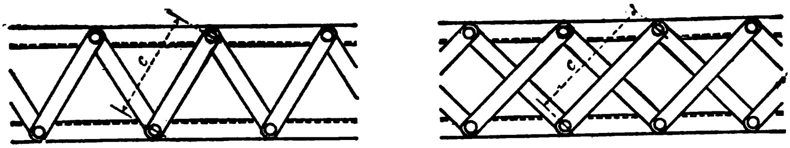
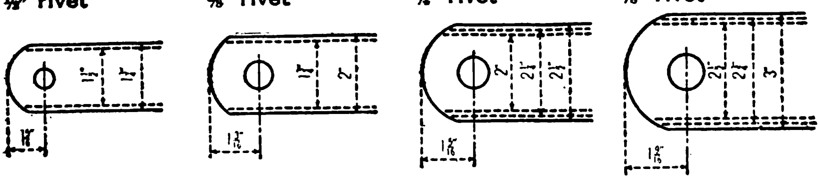


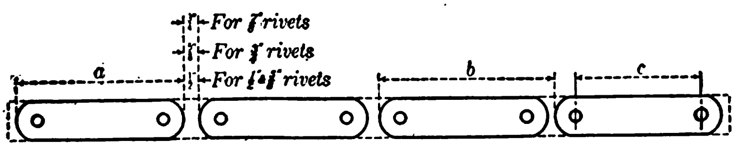
MAXIMUM DISTANCE c IN FEET AND INCHES

FOR GIVEN THICKNESS t OF LACE BAR

THICKNESS OF BAR t INCHES	SINGLE LACING		DOUBLE LACING		THICKNESS OF BAR t INCHES
	$t=\frac{c}{40}$	$t=\frac{c}{50}$	$t=\frac{c}{60}$	$t=\frac{c}{75}$	
⅝	2-1	2-7¼	3-1½	3-10⅞	⅝
9/16	1-10½	2-4	2-9¾	3-6¼	9/16
½	1-8	2-1	2-6	3-1½	½
7/16	1-5½	1-9⅞	2-2¼	2-8¾	7/16
⅜	1-3	1-6¾	1-10½	2-4	⅜
5/16	1-0½	1-3½	1-6¾	1-11½	5/16
¼	0-10	1-0½	1-3	1-6¾	¼

½″ rivet ⅝″ rivet ¾″ rivet ⅞″ rivet

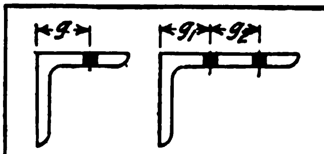




DISTANCE IN INCHES TO BE ADDED TO LENGTH c

WIDTH OF BAR INCHES	FOR FINISHED LENGTH a				FOR ORDERED LENGTH b				WIDTH OF BAR INCHES
	DIAMETER OF RIVET				DIAMETER OF RIVET				
	½″	⅝″	¾″	⅞″	½″	⅝″	¾″	⅞″	
3				3⅛				3⅞	3
2¾				3⅛				3⅞	2¾
2½			2⅝	3⅛			3⅛	3⅞	2½
2¼			2⅝				3⅛		2¼
2		2⅜	2⅝			2⅞	3⅛		2
1¾	1⅞	2⅜			2⅜	2⅞			1¾
1½	1⅞				2⅜				1½

FIG. 228. STANDARDS FOR LACING BARS.

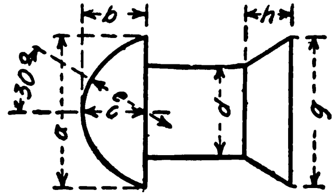


GAGES in inches

Leg	Gage g	Max. Rivet	Leg	Gage g_1	Gage g_2	Max. Rivet
8	$4\frac{1}{2}$	$\frac{7}{8}$	8	3	3	$\frac{7}{8}$
7	4	$\frac{7}{8}$	7	$2\frac{1}{2}$	3	$\frac{7}{8}$
6	$3\frac{1}{2}$	$\frac{7}{8}$	6	$2\frac{1}{4}$	$2\frac{1}{4}$	$\frac{7}{8}$
5	3	$\frac{7}{8}$	5	2	$1\frac{3}{4}$	$\frac{7}{8}$
4	$2\frac{1}{2}$	$\frac{7}{8}$	When 6" L exceeds $\frac{3}{4}$"			
$3\frac{1}{2}$	2	$\frac{7}{8}$	6	$2\frac{1}{2}$	2	$\frac{7}{8}$
3	$1\frac{3}{4}$	$\frac{7}{8}$				
$2\frac{3}{4}$	$1\frac{5}{8}$	$\frac{3}{4}$				
$2\frac{1}{2}$	$1\frac{3}{8}$	$\frac{3}{4}$				
$2\frac{1}{4}$	$1\frac{1}{4}$	$\frac{5}{8}$				
2	$1\frac{1}{8}$	$\frac{5}{8}$				
$1\frac{3}{4}$	1	$\frac{1}{2}$				
$1\frac{1}{2}$	$\frac{7}{8}$	$\frac{3}{8}$				
$1\frac{3}{8}$	$\frac{7}{8}$	$\frac{3}{8}$				
$1\frac{1}{4}$	$\frac{3}{4}$	$\frac{3}{8}$				
1	$\frac{5}{8}$	$\frac{1}{4}$				

MINIMUM RIVET SPACING

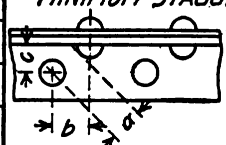
Size of Rivet inches	Min. Distance inches
1	3
$\frac{7}{8}$	$2\frac{5}{8}$
$\frac{3}{4}$	$2\frac{1}{4}$
$\frac{5}{8}$	2
$\frac{1}{2}$	$1\frac{3}{4}$
$\frac{3}{8}$	$1\frac{1}{2}$
$\frac{1}{4}$	1



PROPORTIONS OF RIVETS in inches

Diameter of Shank d	Full Head: Diameter a	Full Head: Height b	Full Head: Radii c	Full Head: Radii e	Countersunk: Diameter g	Countersunk: Depth h
1	$1\frac{5}{8}$	$\frac{11}{16}$	$\frac{11}{16}$	$1\frac{1}{32}$	$1\frac{9}{16}$	$\frac{1}{2}$
$\frac{7}{8}$	$1\frac{7}{16}$	$\frac{39}{64}$	$\frac{39}{64}$	$\frac{59}{64}$	$1\frac{3}{8}$	$\frac{7}{16}$
$\frac{3}{4}$	$1\frac{1}{4}$	$\frac{17}{32}$	$\frac{17}{32}$	$\frac{51}{64}$	$1\frac{3}{16}$	$\frac{3}{8}$
$\frac{5}{8}$	$1\frac{1}{16}$	$\frac{29}{64}$	$\frac{29}{64}$	$\frac{43}{64}$	1	$\frac{5}{16}$
$\frac{1}{2}$	$\frac{7}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{9}{16}$	$\frac{25}{32}$	$\frac{1}{4}$
$\frac{3}{8}$	$\frac{11}{16}$	$\frac{19}{64}$	$\frac{19}{64}$	$\frac{7}{16}$	$\frac{19}{32}$	$\frac{3}{16}$

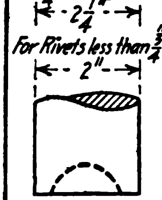
MINIMUM STAGGER FOR RIVETS



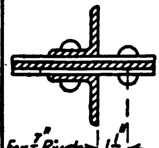
c inches	b in inches: For $\frac{3}{4}$" Rivet $a=1\frac{1}{8}$"	b in inches: For $\frac{7}{8}$" Rivet $a=1\frac{1}{4}$"
$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{1}{2}$
$1\frac{3}{16}$	$1\frac{3}{16}$	$1\frac{7}{16}$
$1\frac{1}{4}$	$1\frac{1}{8}$	$1\frac{3}{8}$

c inches	b in inches: For $\frac{3}{4}$" Rivet $a=1\frac{1}{8}$"	b in inches: For $\frac{7}{8}$" Rivet $a=1\frac{1}{4}$"
$1\frac{5}{16}$	$1\frac{1}{16}$	$1\frac{5}{16}$
$1\frac{3}{8}$	$\frac{15}{16}$	$1\frac{1}{4}$
$1\frac{7}{16}$	$\frac{7}{8}$	$1\frac{3}{16}$
$1\frac{1}{2}$	$\frac{3}{4}$	$1\frac{1}{8}$
$1\frac{9}{16}$	$\frac{9}{16}$	1
$1\frac{5}{8}$	$\frac{3}{8}$	$\frac{15}{16}$
$1\frac{11}{16}$	0	$\frac{13}{16}$
$1\frac{3}{4}$		$\frac{5}{8}$
$1\frac{13}{16}$		$\frac{7}{16}$
$1\frac{7}{8}$		0

MINIMUM BUTTON SETS



STANDARD CLEARANCE FOR RIVETING



RIVETS IN CRIMPED Ls

Distance b should $= 1\frac{1}{2}" + 2 \times$ thickness chord Ls, but never less than 2"

FIG. 229. STANDARDS FOR RIVETS AND RIVETING.

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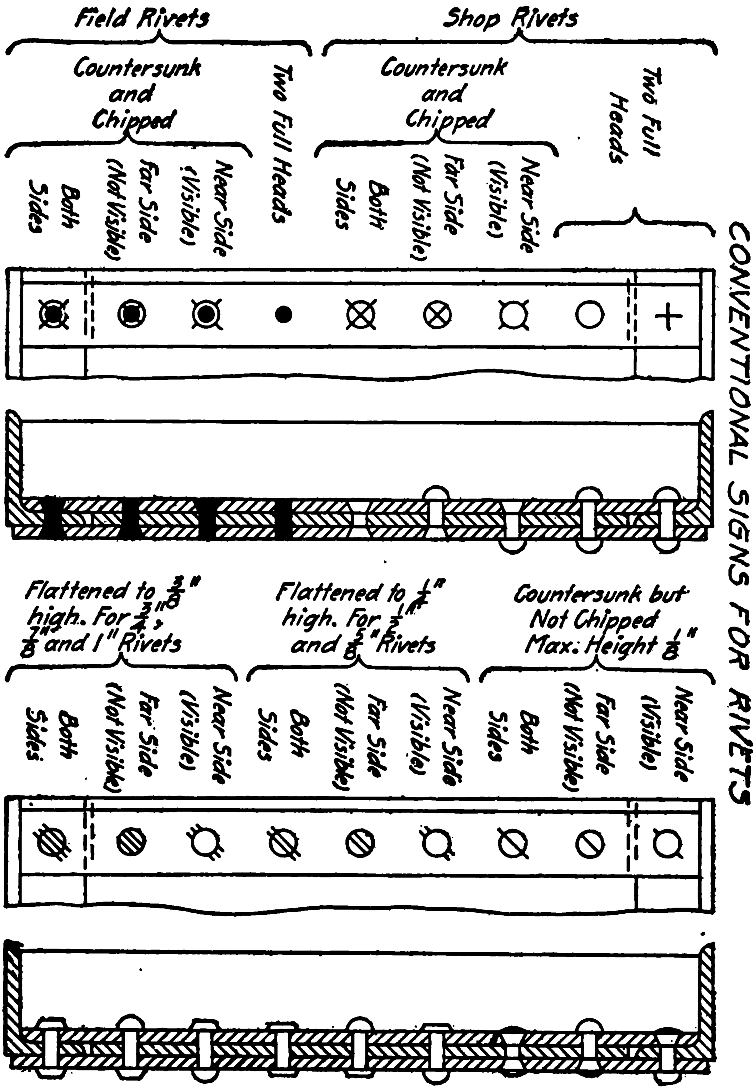

FIG. 230. CONVENTIONAL SIGNS FOR RIVETS AND RIVETING.

Standards for Rivets.—The American Bridge Company's standards for rivet sizes and spacing are given in Fig. 229, while the conventional signs for rivets are given in Fig. 230.

Allowable Stresses in Rivets.—The allowable bearing and shearing stresses for different allowable stresses are given in Tab' XXXI.

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TABLE XXXI.

Shearing and Bearing Value of Rivets.

Values Above or to Right of Upper Zigzag Lines are Greater Than Double Shear. Values Below or to Left of Lower Zigzag Lines are Less Than Single Shear.

Diam. of Rivet. Frac.	Diam. of Rivet. Dec'l.	Area in Sq. Ins.	Single Shear at 6,000 Pounds	Bearing Value for Different Thickness of Plate at 12,000 Pounds per Square Inch. 1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
3/8	.375	.1104	660	1,130	1,410	1,690										
1/2	.500	.1963	1,180	1,500	1,880	2,250	2,630	3,000								
5/8	.625	.3068	1,840	1,880	2,340	2,810	3,280	3,750	4,220	4,690						
3/4	.750	.4418	2,650	2,250	2,810	3,380	3,940	4,500	5,160	5,630	6,190	6,750				
7/8	.875	.6013	3,610	2,630	3,280	3,940	4,590	5,250	5,910	6,560	7,220	7,880	8,530	9,190	9,840	
1	1.000	.7854	4,710	3,000	3,750	4,500	5,250	6,000	6,750	7,500	8,250	9,000	9,750	10,500	11,250	12,000

Diam. of Rivet. Frac.	Diam. of Rivet. Dec'l.	Area in Sq. Ins.	Single Shear at 7,500 Pounds	Bearing Value for Different Thickness of Plate at 15,000 Pounds per Square Inch. 1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
3/8	.375	.1104	830	1,410	1,760	2,110										
1/2	.500	.1963	1,470	1,880	2,340	2,810	3,280	3,750								
5/8	.625	.3068	2,300	2,340	2,930	3,520	4,100	4,690	5,280	5,860						
3/4	.750	.4418	3,310	2,810	3,520	4,220	4,920	5,630	6,330	7,030	7,720	8,440				
7/8	.875	.6013	4,510	3,280	4,100	4,920	5,740	6,560	7,380	8,200	9,030	9,850	10,670	11,480	12,300	
1	1.000	.7854	5,890	3,750	4,690	5,620	6,560	7,500	8,440	9,380	10,310	11,250	12,190	13,130	14,060	15,000

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TABLE XXXI.—*Continued.*

Shearing and Bearing Value of Rivets.

Values Above or to Right of Upper Zigzag Lines are Greater Than Double Shear. Values Below or to Left of Lower Zigzag Lines are Less Than Single Shear.

Diam. of Rivet.		Area in Sq. Ins.	Single Shear at 11,000 Pounds.	Bearing Value for Different Thickness of Plate at 22,000 Pounds per Square Inch.												
Frac.	Dec'l.			1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
3/8	.375	.1104	1,210	2,060	2,580	3,090										
1/2	.500	.1963	2,160	2,750	3,440	4,130	4,820	5,500								
5/8	.625	.3068	3,370	3,440	4,300	5,160	6,020	6,880	7,740	8,600						
3/4	.750	.4418	4,860	4,130	5,160	6,190	7,220	8,250	9,280	10,320	11,340	12,380				
7/8	.875	.6013	6,610	4,810	6,020	7,220	8,430	9,630	10,840	12,040	13,240	14,440	15,640	16,840	18,050	
1	1.000	.7854	8,640	5,500	6,880	8,250	9,630	11,000	12,380	13,750	15,130	16,500	17,880	19,520	20,630	22,000

Diam. of Rivet.		Area in Sq. Ins.	Single Shear at 12,000 Pounds.	Bearing Value for Different Thickness of Plate at 24,000 Pounds per Square Inch.												
Frac.	Dec'l.			1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
3/8	.375	.1104	1,320	2,250	2,810	3,380										
1/2	.500	.1963	2,360	3,000	3,750	4,500	5,250	6,000								
5/8	.625	.3068	3,680	3,750	4,690	5,620	6,560	7,500	8,440	9,370						
3/4	.750	.4418	5,300	4,500	5,620	6,750	7,870	9,000	10,120	11,250	12,370	13,500				
7/8	.875	.6013	7,220	5,250	6,560	7,870	9,190	10,500	11,810	13,120	14,440	15,750	17,060	18,370		
1	1.000	.7854	9,430	6,000	7,500	9,000	10,500	12,000	13,500	15,000	16,500	18,000	19,500	21,000	22,500	24,000

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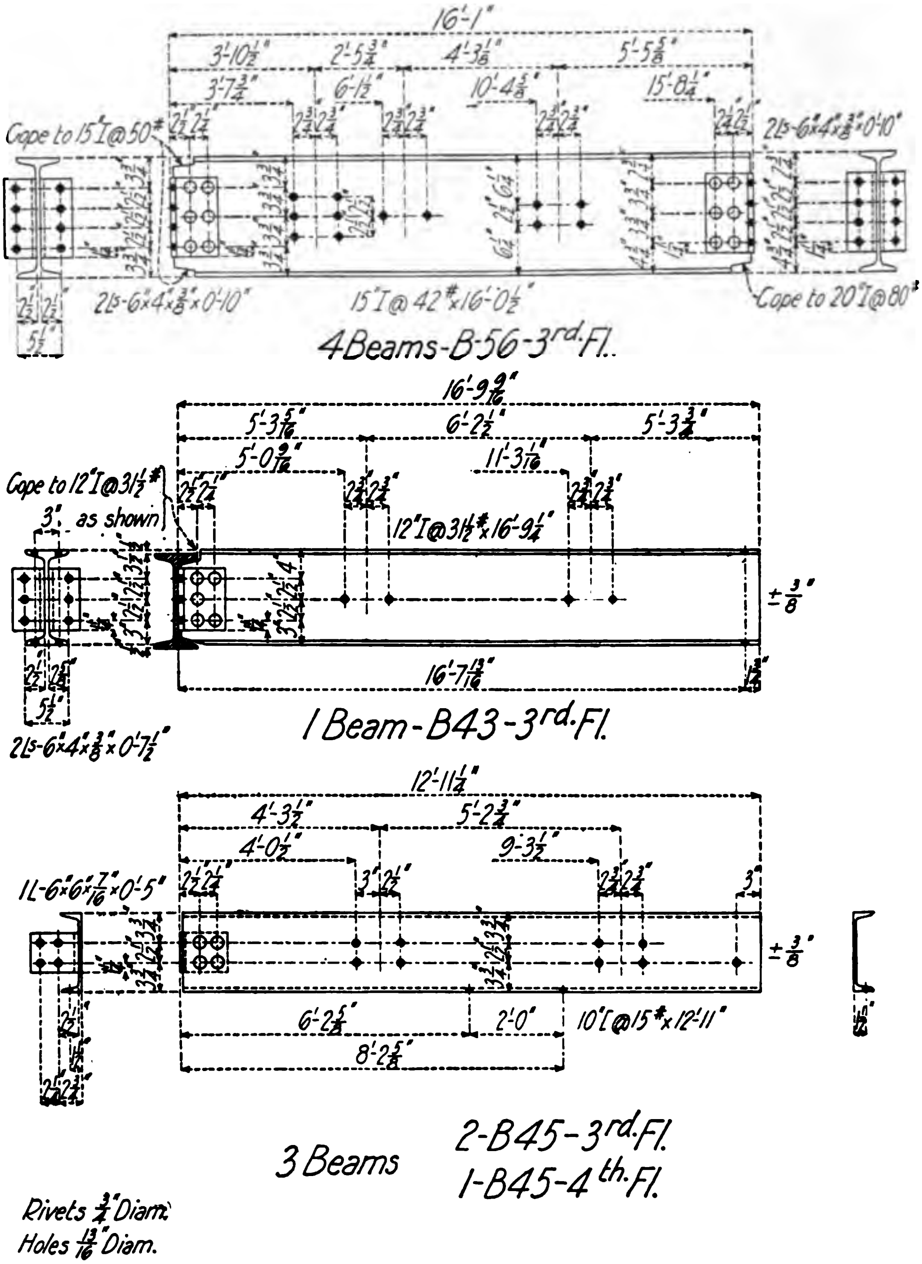

FIG. 231. STANDARD DETAILS FOR ROLLED I-BEAMS AND CHANNELS.

Details of Rolled Beams.—Standard details for rolled I-Beams and channels are given in Fig. 231 and Fig. 232. Standard connection angles with the rivet spacing are given in Tables XXXII to XXXIV.

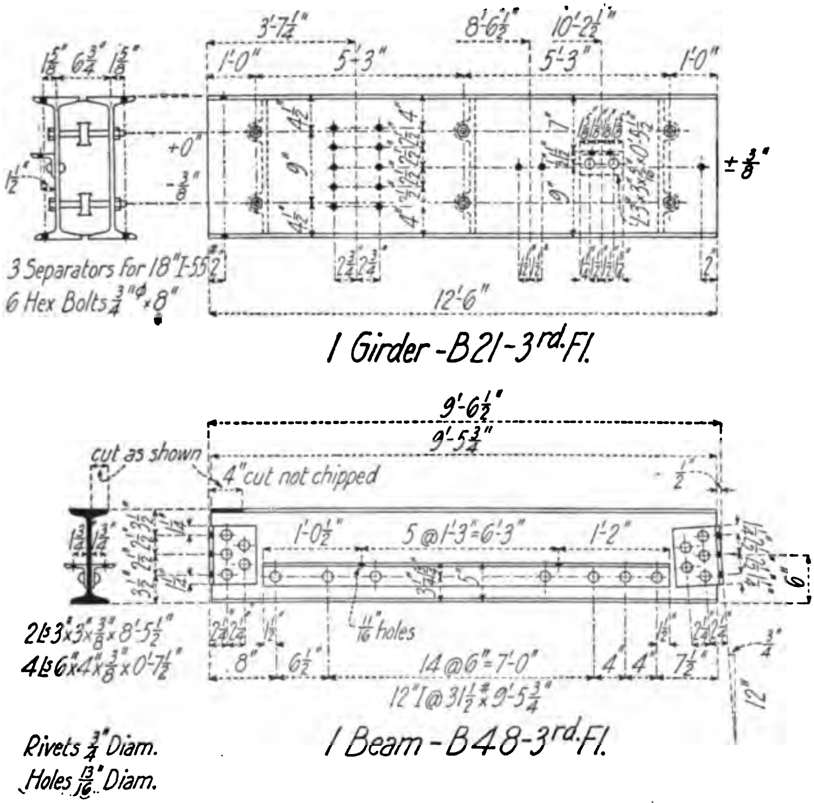

FIG. 232. STANDARD DETAILS FOR ROLLED I-BEAMS AND CHANNELS.

Details of Columns.—Details of a rolled H-column, Bethlehem H-beam, are given in Fig. 233, while details of a built H-column are given in Fig. 234. Details of a column for the steel head frame for the Copper Queen Mining Co. are given in Fig. 235.

Hand-books.—Properties of sections and other useful data are given in manufacturer's hand-books of which the "Carnegie," issued by the Carnegie Steel Company, and the "Cambria Steel," issued by the Cambria Steel Company, are the best known. These books can be

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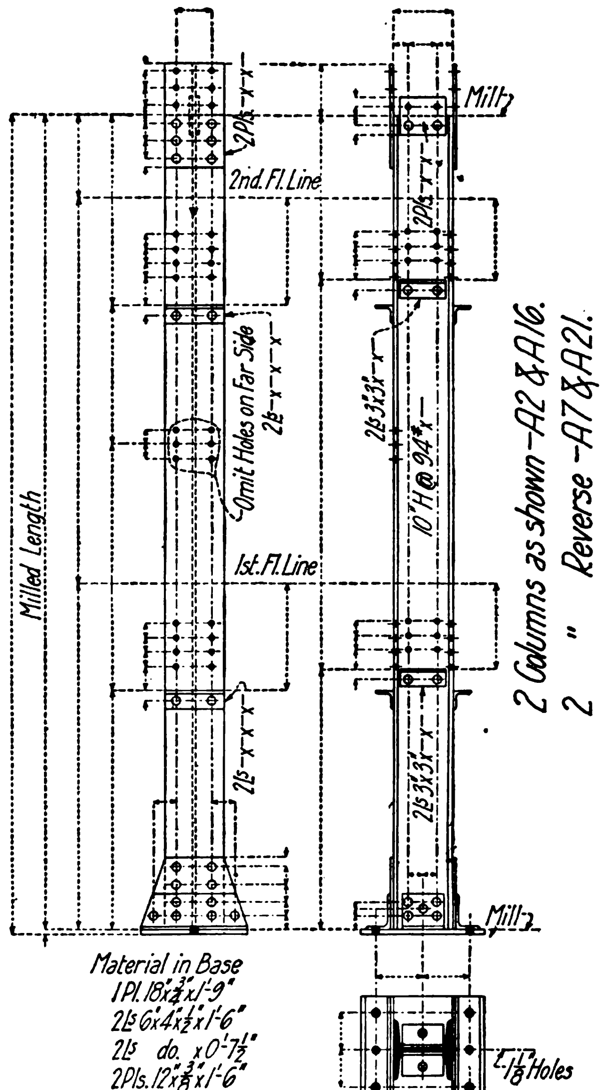

FIG. 233. DETAILS FOR A BETHLEHEM H-COLUMN.

purchased by students and engineers for 50 cents each, and are invaluable to the structural steel designer.

Structural Tables.—Tables for use in structural steel design not ordinarily found in hand-books are given in Table XXXII to Table XLVI.

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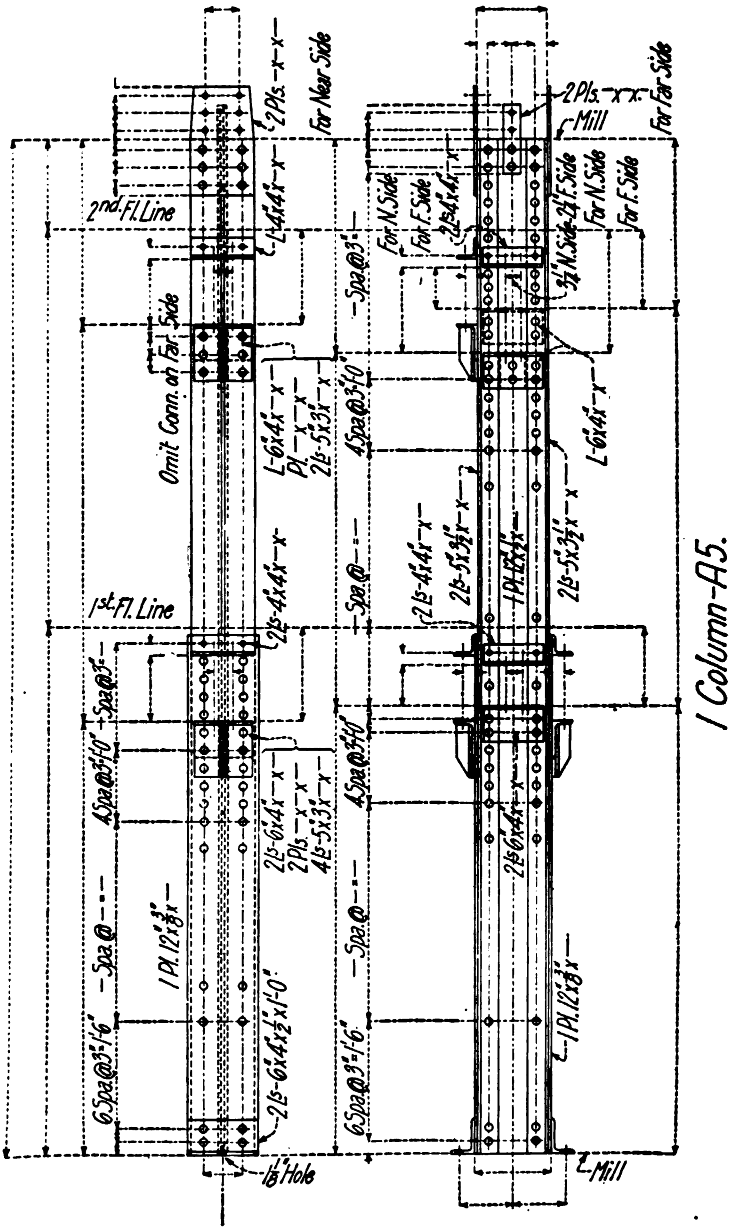

FIG. 234. DETAILS FOR A BUILT H-COLUMN.

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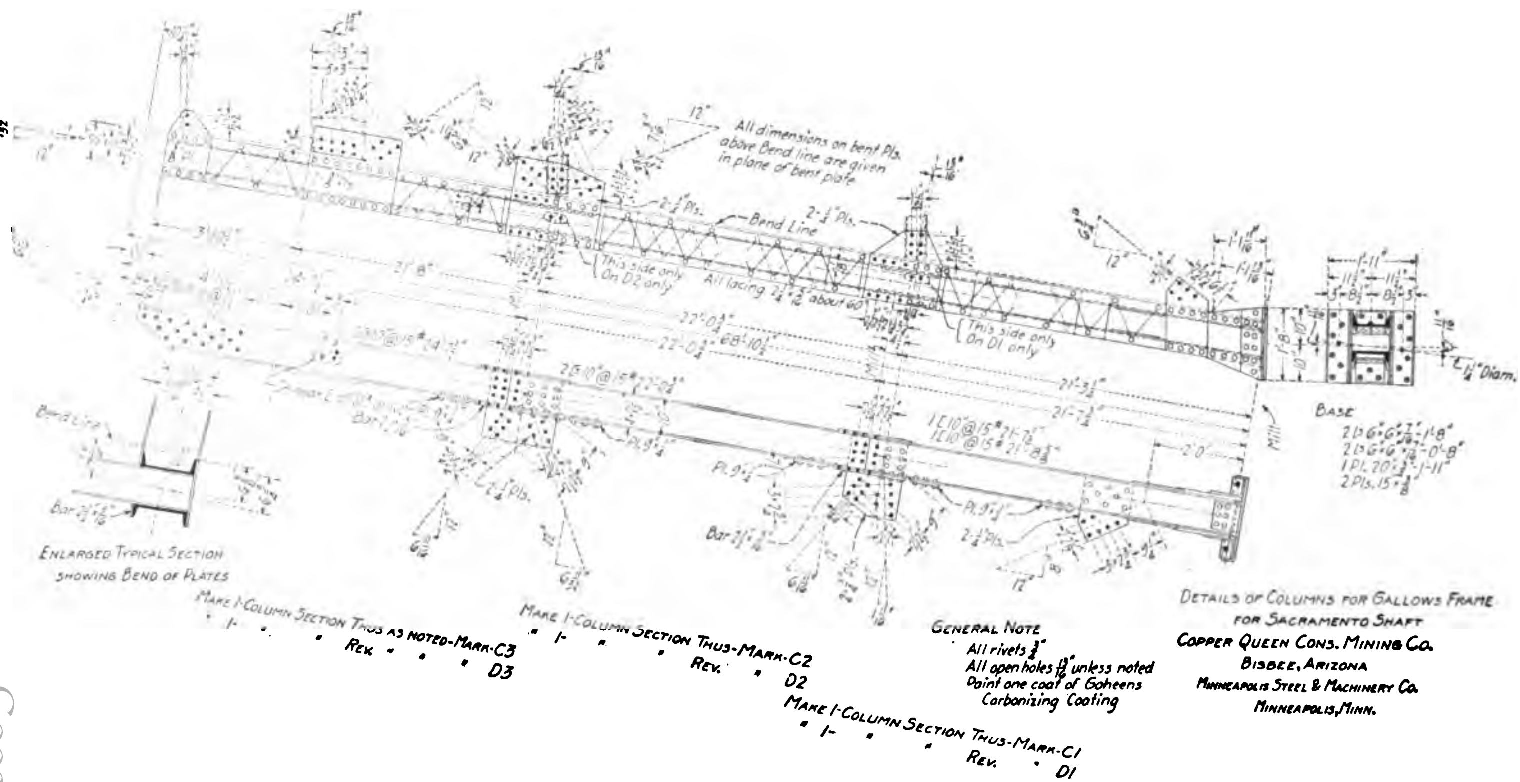
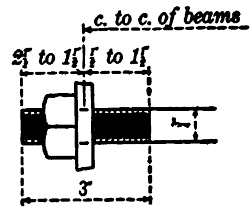

All dimensions on bent Pls. above Bend line are given in plane of bent plate
Bend Line
This side only On D2 only
This side only On D1 only
All lacing $2\frac{1}{4}$" x $\frac{5}{16}$" about 60°
Enlarged Typical Section showing Bend of Plates
Base
Make 1-Column Section Thus as noted-Mark-C3
" 1- " " Rev. " " D3
Make 1-Column Section Thus-Mark-C2
" 1- " " Rev. " D2
Make 1-Column Section Thus-Mark-C1
" 1- " " Rev. " D1
General Note
All rivets $\frac{3}{4}$"
All open holes $\frac{13}{16}$" unless noted
Paint one coat of Goheens Carbonizing Coating
Details of Columns for Gallows Frame for Sacramento Shaft
Copper Queen Cons. Mining Co.
Bisbee, Arizona
Minneapolis Steel & Machinery Co.
Minneapolis, Minn.

FIG. 235.

TIE RODS

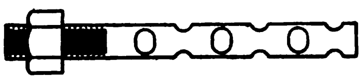


UPPER FIGURES IN TABLE GIVE LENGTHS IN FEET AND INCHES. Lower figures give weights in pounds of one rod and two nuts.

LENGTHS AND WEIGHTS OF ¾" TIE RODS
FOR DIFFERENT DISTANCES CENTER TO CENTER OF BEAMS

FEET	0"	1"2"3"	4"5"6"	7"8"9"	10"11"
1			1-9 3.4	2-0 3.8	2-3 4.2
2	2-3 4.2	2-6 4.5	2-9 4.9	3-0 5.3	3-3 5.7
3	3-3 5.7	3-6 6.0	3-9 6.4	4-0 6.8	4-3 7.2
4	4-3 7.2	4-6 7.5	4-9 7.9	5-0 8.3	5-3 8.7
5	5-3 8.7	5-6 9.0	5-9 9.4	6-0 9.8	6-3 10.2
6	6-3 10.2	6-6 10.5	6-9 10.9	7-0 11.3	7-3 11.7
7	7-3 11.7	7-6 12.0	7-9 12.4	8-0 12.8	8-3 13.2
8	8-3 13.2	8-6 13.5	8-9 13.9	9-0 14.3	9-3 14.7

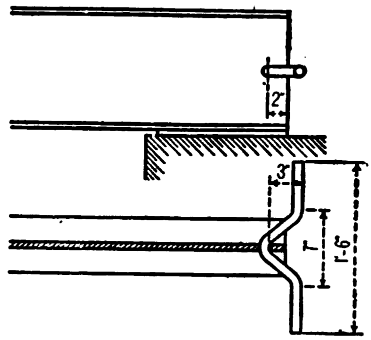
SWEDGE BOLT



DIAMETER INCHES	LENGTH FEET & INS.	WEIGHT INCLUDING NUT POUNDS
¾	0-9	2
⅞	1-0	3
1	1-0	4
1¼	1-3	7

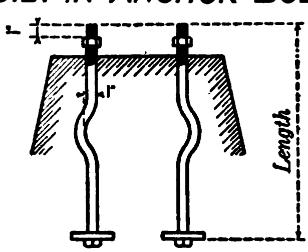
Punch holes ⅛" larger than diameter of bolt.

GOVERNMENT ANCHOR



¾" Rod 1'-9" long Weight 3 pounds

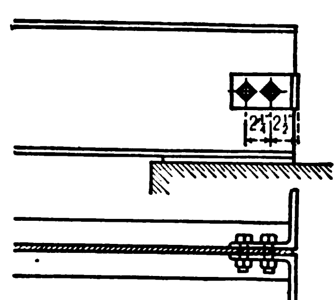
BUILT-IN ANCHOR BOLTS



DIAMETER INCHES	LENGTH FEET & INS.	WASHER INCHES	WEIGHT EACH WITH NUT AND WASHER POUNDS
¾	2-0	4 x ⅜ x 4	6
⅞	2-6	4 x ⅜ x 4	8
1	3-0	6 x ⅜ x 6	14
1¼	4-0	6 x ⅜ x 6	24

When center to center of anchors is less than width of washer, use washer with two holes.

ANGLE ANCHOR



2 Angles 6"x 6"x 7/16"-0'-3" and two ¾" bolts.
Weight with bolts 10 pounds.

FIG. 236.

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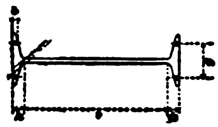
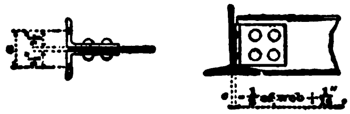
TABLE XXXII.

Carnegie I-Beams.





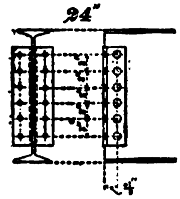
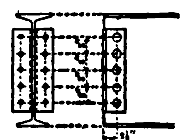
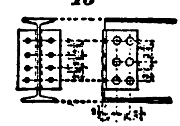
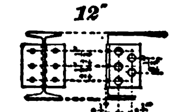
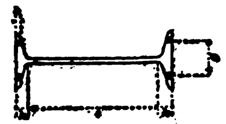
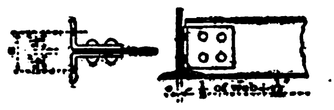
I	Weight per Foot	Flange	Web	Gauge g	Tang't t	Dist. k	Grip b	Max. Rivet or Bolt	Wall Bearing	Wall Pl.	Standard Framing	Dist. a	Dist. c	Weight per Foot	I
Inches	Pounds	Inches	Inches	Inches	Inches	Inches	Inches	Inches	Inches			Inches	Inches	Pounds	Inches
24	100.0	7 1/4	3/4	4	20 3/4	1 5/8	7/8	1	16	16"×1"×1'-4" 73#	24"  2-L^s 4"×4"×5/16"×1'-6" Wt. 37#	5 3/4	7/16	100.0	24
	95.0	7 3/16	11/16	"	"	"	"					5 11/16	13/32	95.0	
	90.0	7 1/8	5/8	"	"	"	"					5 5/8	3/8	90.0	
	85.0	7 1/16	9/16	"	"	"	"					5 9/16	3/8	85.0	
	80.0	7	1/2	"	"	"	"					5 1/2	5/16	80.0	
20	100.0	7 9/32	7/8	"	16 1/2	1 3/4	15/16	7/8	16	16"×1"×1'-4" 73#		5 7/8	1/2	100.0	20
	95.0	7 7/32	13/16	"	"	"	"					5 13/16	1/2	95.0	
	90.0	7 1/8	23/32	"	"	"	"					5 3/4	7/16	90.0	
	85.0	7 1/16	21/32	"	"	"	"					5 11/16	7/16	85.0	
	80.0	7	19/32	"	"	"	"				18" and 20"  2-L^s 4"×4"×5/16"×1'-3" Wt. 31#	5 5/8	3/8	80.0	
	75.0	6 13/32	21/32	"	17	1 1/2	3/4					5 13/16	3/8	75.0	
	70.0	6 5/16	9/16	"	"	"	"					5 9/16	3/8	70.0	
	65.0	6 1/4	1/2	"	"	"	"					5 1/2	5/16	65.0	
18	70.0	6 1/4	23/32	3 3/4	15 1/4	1 3/8	5/8	7/8	16	16"×1"×1'-4" 73#		5 3/4	7/16	70.0	18
	65.0	6 3/16	5/8	"	"	"	"					5 5/8	3/8	65.0	
	60.0	6 3/32	17/32	"	"	"	"					5 9/16	3/8	60.0	
	55.0	6	7/16	"	"	"	"					5 1/2	5/16	55.0	
15	100.0	6 25/32	1 3/16	"	11	2	1	7/8	12	12"×3/4"×1'-4" 41#	15"  2-L^s 6"×4"×5/16"×0'-10" Wt. 27#	6 3/16	11/16	100.0	15
	95.0	6 21/32	1 1/16	"	"	"	"					6 1/16	5/8	95.0	
	90.0	6 9/16	31/32	"	"	"	"					6	9/16	90.0	
	85.0	6 15/32	7/8	"	"	"	"					5 7/8	1/2	85.0	
	80.0	6 13/32	13/16	"	"	"	"					5 13/16	1/2	80.0	
	75.0	6 9/32	7/8	"	11 3/4	1 5/8	3/4	3/4				5 7/8	1/2	75.0	
	70.0	6 3/16	25/32	"	"	"	"					5 3/4	7/16	70.0	
	65.0	6 3/32	11/16	"	"	"	"					5 11/16	7/16	65.0	
	60.0	6	19/32	"	"	"	"					5 5/8	3/8	60.0	
	55.0	5 3/4	21/32	3 1/2	12 1/2	1 1/4	9/16					5 5/8	3/8	55.0	
	50.0	5 5/8	17/32	"	"	"	"					5 9/16	3/8	50.0	
	45.0	5 17/32	7/16	"	"	"	"					5 1/2	5/16	45.0	
	42.0	5 1/2	13/32	"	"	"	"					5 7/16	1/4	42.0	
12	55.0	5 5/8	13/16	3 1/2	9 1/4	1 3/8	11/16	3/4	12	12"×3/4"×1'-0" 31#	12"  2-L^s 6"×4"×5/16"×0'-7 1/2" Wt. 20#	5 13/16	1/2	55.0	12
	50.0	5 1/2	11/16	"	"	"	"					5 11/16	7/16	50.0	
	45.0	5 3/8	9/16	3	"	"	"					5 9/16	3/8	45.0	
	40.0	5 1/4	7/16	"	"	"	"					5 1/2	5/16	40.0	
	35.0	5 3/32	7/16	"	9 3/4	1 1/8	5/8					5 7/16	5/16	35.0	
	31.5	5	11/32	"	"	"	"					5 3/8	1/4	31.5	

TABLE XXXIII.

Carnegie I-Beams.

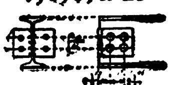




I	Weight per foot	Flange	Web	Gauge g	Tangent t	Dist. h	Grip b	Max. rivet or bolt	Wall bearing	Wall plate	Dist. a	Dist. c	Weight per foot	I
Inches	Pounds	Inches	Inches	Inches	Inches	Inches	Inches	Inches	Inches		Inches	Inches	Pounds	Inches
10	40.0	5 [illegible]	$\frac{1}{2}$	3	8	1	[illegible]	$\frac{3}{4}$	8	8" x $\frac{1}{2}$" x 1'-0" 17#	5 [illegible]	[illegible]	40.0	10
	35.0	4 [illegible]	[illegible]	2 [illegible]	"	"	"				5 [illegible]	[illegible]	35.0	
	30.0	4 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	30.0	
	25.0	4 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	25.0	
9	35.0	4 [illegible]	[illegible]	"	7	"	"	$\frac{3}{4}$	8		5 [illegible]	[illegible]	35.0	9
	30.0	4 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	30.0	
	25.0	4 [illegible]	[illegible]	2 $\frac{1}{2}$	"	"	"				5 [illegible]	[illegible]	25.0	
	21.0	4 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	21.0	
8	25.5	4 [illegible]	[illegible]	"	6 $\frac{1}{2}$	[illegible]	"	$\frac{3}{4}$	8	8" x [illegible]" x 0'-8" 12#	5 [illegible]	[illegible]	25.5	8
	23.0	4 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	23.0	
	20.5	4 [illegible]	[illegible]	2 [illegible]	"	"	"				5 [illegible]	[illegible]	20.5	
	18.0	4	[illegible]	"	"	"	"				5 [illegible]	[illegible]	18.0	
7	20.0	3 [illegible]	[illegible]	"	5 $\frac{1}{2}$	"	$\frac{1}{2}$	$\frac{5}{8}$	8		5 [illegible]	[illegible]	20.0	7
	17.5	3 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	17.5	
	15.0	3 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	15.0	
6	17.25	3 [illegible]	[illegible]	2	4 $\frac{1}{2}$	$\frac{3}{4}$	"	$\frac{5}{8}$	6	[illegible]	5 [illegible]	[illegible]	17.25	6
	14.75	3 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	14.75	
	12.25	3 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	12.25	
5	14.75	3 [illegible]	[illegible]	1 $\frac{3}{4}$	3 [illegible]	[illegible]	[illegible]	$\frac{1}{2}$	6	[illegible]" x [illegible]" x 0'-8"	5 [illegible]	[illegible]	14.75	5
	12.25	3 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	12.25	
	9.75	3	[illegible]	"	"	"	"				5 [illegible]	[illegible]	9.75	
4	10.5	2 [illegible]	[illegible]	1 $\frac{1}{2}$	2 $\frac{1}{2}$	"	"	$\frac{1}{2}$	6		5 [illegible]	[illegible]	10.5	4
	9.5	2 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	9.5	
	8.5	2 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	8.5	
	7.5	2 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	7.5	
3	7.5	2 [illegible]	[illegible]	1 $\frac{1}{4}$	1 [illegible]	"	$\frac{1}{4}$	$\frac{1}{2}$	6		5 [illegible]	[illegible]	7.5	3
	6.5	2 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	6.5	
	5.5	2 [illegible]	[illegible]	"	"	"	"				5 [illegible]	[illegible]	5.5	

STANDARD FRAMING

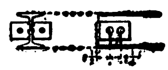
7", 8", 9", & 10"



2-L's 6" x 4" x [illegible]" x 0'-8½" Wt. 14#

5" & 6"

2-L's 6" x 4" x [illegible]" x 0'-3" for 6" Wt. 6#
2-L's " [illegible] for 5" Wt. 7#



3" & 4"

2-L's 6" x 4" x $\frac{7}{16}$" x 0'-3" Wt. 6#

All rivets in standard framing angles are $\frac{3}{4}$" diam.
Weights of " " " include weight of shop rivets only.
When beams frame opposite each other into another beam with web thickness less than $\frac{5}{16}$" or where beams of short span lengths are loaded to their full capacity, it may be necessary to use framing angles of greater strength than the standards.
See table below for minimum span lengths.

I	Weight	Span in ft.	I	Weight	Span in ft.	I	Weight	Span in ft.	I	Weight	Span in ft.	I	Weight	Span in ft.	I	Weight	Span in ft.	I	Weight	Span in ft.
24	80.0	22.0				15	80.0	20.0							8	18.0	5.5	5	9.75	4.0
20	80.0	22.0	18	55.0	14.0	"	60.0	15.5	12	40.0	11.5	10	25.0	9.0	7	15.0	4.0	4	7.5	3.0
"	65.0	18.0				"	42.0	11.0	"	31.5	9.0	9	21.0	7.0	6	12.25	6.0	3	5.5	2.0

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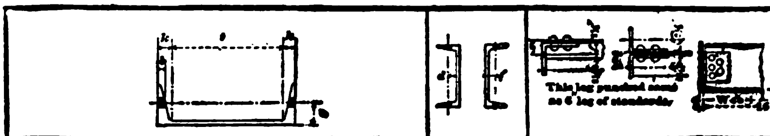
TABLE XXXIV.

CARNEGIE CHANNELS.



	Weight per Foot	Flange	Web	Gauge g	Tang't. t	Dist. k	Grip b	Diam. Rivet or Bolt	Dist. d	Gauge f	Dist. s	Dist. q	Weight per Foot	
15	55.00	3 13/16	13/16	2 1/4	12 1/4	1 1/16	5/8	7/8	1 1/4	2 7/16	3 3/8	7/8	55.00	15
	50.00	3 23/32	23/32	"	"	"	"		"	2 5/16	3 1/4	13/16	50.00	
	45.00	3 5/8	5/8	2	"	"	"		"	2 1/4	3 1/8	3/4	45.00	
	40.00	3 17/32	17/32	"	"	"	"		"	2 3/16	3	11/16	40.00	
	35.00	3 7/16	7/16	"	"	"	"		"	2 1/8	2 7/8	5/8	35.00	
	33.00	3 13/32	13/32	"	"	"	"		"	2 1/16	2 13/16	1/2	33.00	
12	40.00	3 13/32	3/4	"	10	1	1/2	3/4	1 3/16	2 1/16	3 1/4	13/16	40.00	12
	35.00	3 5/16	5/8	"	"	"	"		"	2 1/16	3 1/8	11/16	35.00	
	30.00	3 3/16	1/2	1 3/4	"	"	"		"	1 15/16	3	5/8	30.00	
	25.00	3 1/16	13/32	"	"	"	"		"	1 7/8	2 7/8	9/16	25.00	
	20.50	2 15/16	9/32	"	"	"	"		"	1 3/4	2 13/16	1/2	20.50	
10	35.00	3 3/16	13/16	"	8 1/4	7/8	"	3/4	1 1/8	2 1/16	3 3/8	7/8	35.00	10
	30.00	3 1/32	11/16	"	"	"	"		"	1 15/16	3 1/4	3/4	30.00	
	25.00	2 7/8	17/32	"	"	"	7/16		"	1 13/16	3 1/8	5/8	25.00	
	20.00	2 3/4	3/8	1 3/4	"	"	"		"	1 5/8	2 7/8	9/16	20.00	
	15.00	2 19/32	1/4	"	"	"	"		"	1 1/2	2 3/4	7/16	15.00	
9	25.00	2 13/16	5/8	"	7 1/4	7/8	"	3/4	1 1/8	1 3/4	3 1/8	11/16	25.00	9
	20.00	2 21/32	7/16	"	"	"	"		"	1 5/8	2 15/16	5/8	20.00	
	15.00	2 1/2	9/32	1 5/8	"	"	"		"	1 9/16	2 13/16	1/2	15.00	
	13.25	2 7/16	7/32	"	"	"	"		"	1 7/16	2 3/4	1/2	13.25	
8	21.25	2 5/8	9/16	1 1/2	6 1/4	"	3/8	3/4	1 1/16	1 11/16	3 1/8	5/8	21.25	8
	18.75	2 17/32	1/2	"	"	"	"		"	1 5/8	3	9/16	18.75	
	16.25	2 7/16	13/32	"	"	"	"		"	1 9/16	2 7/8	1/2	16.25	
	13.75	2 11/32	5/16	1 3/8	"	"	"		"	1 1/2	2 13/16	1/2	13.75	
	11.25	2 1/4	7/32	"	"	"	"		"	1 7/16	2 3/4	7/16	11.25	
7	19.75	2 1/2	5/8	1 1/2	5 1/4	3/4	"	5/8	1 1/16	1 11/16	3 1/8	11/16	19.75	7
	17.25	2 13/32	17/32	"	"	"	"		"	1 9/16	3	9/16	17.25	
	14.75	2 5/16	7/16	"	"	"	"		"	1 1/2	2 15/16	1/2	14.75	
	12.25	2 3/16	5/16	1 1/4	"	"	"		"	1 3/8	2 13/16	1/2	12.25	
	9.75	2 3/32	7/32	"	"	"	"		"	1 5/16	2 3/4	3/8	9.75	
6	15.50	2 9/32	9/16	"	4 1/2	"	"	5/8	15/16	1 1/2	3 1/16	5/8	15.50	6
	13.00	2 5/32	7/16	"	"	"	"		"	1 3/8	2 15/16	1/2	13.00	
	10.50	2 1/32	5/16	"	"	"	"		"	1 1/4	2 13/16	1/2	10.50	
	8.00	1 29/32	3/16	1 1/8	"	"	"		"	1 1/8	2 11/16	3/8	8.00	
5	11.50	2 1/32	15/32	"	3 1/2	5/8	5/16	1/2	7/8	1 3/16	3	5/16	11.50	5
	9.00	1 29/32	11/32	"	"	"	"		"	1 3/16	2 13/16	1/2	9.00	
	6.50	1 3/4	3/16	"	"	"	"		"	1 1/16	2 11/16	3/8	6.50	
4	7.25	1 23/32	11/32	1	2 1/2	5/8	"	1/2	3/4	1 1/16	2 13/16	1/2	7.25	4
	6.25	1 21/32	1/4	"	"	"	"		"	1	2 3/4	5/16	6.25	
	5.25	1 19/32	3/16	"	"	"	"		"	15/16	2 11/16	3/8	5.25	
3	6.00	1 5/8	3/8	7/8	1 3/4	5/8	1/4	1/2	5/8	1	2 7/8	7/16	6.00	3
	5.00	1 1/2	1/4	"	"	"	"		"	7/8	2 3/4	5/16	5.00	
	4.00	1 13/32	3/16	"	"	"	"		"	13/16	2 11/16	3/8	4.00	

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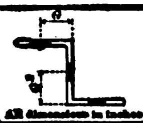
TABLE XXXV.

Carnegie Z-Bars.



All dimensions in inches

Nominal Size	Thickness	Actual Size Flanges & Web	Weight per Foot	Area in Sq. Inches	Gauge g	Max. Rivets g	Max. Rivets g¹	Gauge g²	Thickness	Nominal Size
	1/4	2 11/16 × 3 × 2 11/16	6.7	1.97	1½	5/8	5/8	1½	1/4	
	5/16	2¾ × 3 1/16 × 2¾	8.4	2.48	"	"	"	"	5/16	
	3/8	2 13/16 × 3 1/8 × 2 13/16	9.7	2.86	"	"	"	"	3/8	
3	7/16	2¾ × 3 1/16 × 2¾	11.4	3.36	"	"	"	"	7/16	3
	1/2	2 11/16 × 3 × 2 11/16	12.5	3.69	"	"	"	"	1/2	
	9/16	2¾ × 3 1/16 × 2¾	14.2	4.18	"	"	"	"	9/16	
	1/4	3 1/16 × 4 × 3 1/16	8.2	2.41	2	3/4	7/8	2	1/4	
	5/16	3 1/8 × 4 1/16 × 3 1/8	10.3	3.03	"	"	"	"	5/16	
	3/8	3 3/16 × 4 1/8 × 3 3/16	12.4	3.66	"	"	"	"	3/8	
	7/16	3 1/16 × 4 × 3 1/16	13.8	4.05	"	"	"	"	7/16	
4	1/2	3 1/8 × 4 1/16 × 3 1/8	15.8	4.66	"	"	"	"	1/2	4
	9/16	3 3/16 × 4 1/8 × 3 3/16	17.9	5.27	"	"	"	"	9/16	
	5/8	3 1/16 × 4 × 3 1/16	18.9	5.55	"	"	"	"	5/8	
	11/16	3 1/8 × 4 1/16 × 3 1/8	20.9	6.14	"	"	"	"	11/16	
	3/4	3 3/16 × 4 1/8 × 3 3/16	22.9	6.75	"	"	"	"	3/4	
	5/16	3¼ × 5 × 3¼	11.6	3.40	2¼	7/8	7/8	2¼	5/16	
	3/8	3 5/16 × 5 1/16 × 3 5/16	13.9	4.10	"	"	"	"	3/8	
	7/16	3⅜ × 5⅛ × 3⅜	16.4	4.81	"	"	"	"	7/16	
	1/2	3¼ × 5 × 3¼	17.8	5.23	"	"	"	"	1/2	
5	9/16	3 5/16 × 5 1/16 × 3 5/16	20.2	5.94	"	"	"	"	9/16	5
	5/8	3⅜ × 5⅛ × 3⅜	22.6	6.64	"	"	"	"	5/8	
	11/16	3¼ × 5 × 3¼	23.7	6.96	"	"	"	"	11/16	
	3/4	3 5/16 × 5 1/16 × 3 5/16	26.0	7.64	"	"	"	"	3/4	
	13/16	3⅜ × 5⅛ × 3⅜	28.3	8.33	"	"	"	"	13/16	
	3/8	3½ × 6 × 3½	15.6	4.59	2½	7/8	7/8	3	3/8	
	7/16	3 9/16 × 6 1/16 × 3 9/16	18.3	5.39	"	"	"	"	7/16	
	1/2	3⅝ × 6⅛ × 3⅝	21.0	6.19	"	"	"	"	1/2	
	9/16	3½ × 6 × 3½	22.7	6.68	"	"	"	"	9/16	
6	5/8	3 9/16 × 6 1/16 × 3 9/16	25.4	7.46	"	"	"	"	5/8	6
	11/16	3⅝ × 6⅛ × 3⅝	28.0	8.25	"	"	"	"	11/16	
	3/4	3½ × 6 × 3½	29.3	8.63	"	"	"	"	3/4	
	13/16	3 9/16 × 6 1/16 × 3 9/16	32.0	9.40	"	"	"	"	13/16	
	7/8	3⅝ × 6⅛ × 3⅝	34.6	10.17	"	"	"	"	7/8	

TABLE XXXVI.

Carnegie Angles.

Weights in pounds per lineal foot.

WEIGHTS OF ANGLES

All dimensions in inches

SIZE	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1	$1\frac{1}{16}$	$1\frac{1}{8}$	SIZE
8 × 8							26.4	29.5	32.7	35.8	38.9	42.0	45.0	48.0	51.0	54.0	56.9	8 × 8
6 × 6					14.8	17.2	19.6	21.9	24.2	26.5	28.7	30.9	33.1	35.3	37.4			6 × 6
*5 × 5					12.3	14.3	16.2	18.1	20.0	21.8	23.6	25.4	27.2	28.9	30.6			*5 × 5
4 × 4				8.2	9.8	11.3	12.8	14.3	15.7	17.1	18.5	19.9						4 × 4
$3\frac{1}{2} \times 3\frac{1}{2}$				7.1	8.5	9.8	11.1	12.3	13.6	14.8	16.0	17.1						$3\frac{1}{2} \times 3\frac{1}{2}$
3 × 3			4.9	6.1	7.2	8.3	9.4	10.4	11.4									3 × 3
*$2\frac{3}{4} \times 2\frac{3}{4}$			4.5	5.5	6.6	7.6	8.5											*$2\frac{3}{4} \times 2\frac{3}{4}$
$2\frac{1}{2} \times 2\frac{1}{2}$		3.1	4.0	5.0	5.9	6.8	7.7											$2\frac{1}{2} \times 2\frac{1}{2}$
*$2\frac{1}{4} \times 2\frac{1}{4}$		2.8	3.7	4.5	5.3	6.1	6.8											*$2\frac{1}{4} \times 2\frac{1}{4}$
2 × 2		2.5	3.2	4.0	4.7	5.3												2 × 2
$1\frac{3}{4} \times 1\frac{3}{4}$		2.1	2.8	3.4	4.0	4.6												$1\frac{3}{4} \times 1\frac{3}{4}$
$1\frac{1}{2} \times 1\frac{1}{2}$	1.2	1.8	2.4	2.9	3.4													$1\frac{1}{2} \times 1\frac{1}{2}$
$1\frac{1}{4} \times 1\frac{1}{4}$	1.0	1.5	1.9	2.4														$1\frac{1}{4} \times 1\frac{1}{4}$
1 × 1	0.8	1.2	1.5															1 × 1
SIZE	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1	$1\frac{1}{16}$	$1\frac{1}{8}$	SIZE
*$7 \times 3\frac{1}{2}$						15.0	17.0	19.0	21.0	23.0	24.9	26.8	28.7	30.5	32.3			*$7 \times 3\frac{1}{2}$
6 × 4					12.3	14.3	16.2	18.1	20.0	21.8	23.6	25.4	27.2	28.9	30.6			6 × 4
$6 \times 3\frac{1}{2}$					11.7	13.5	15.3	17.1	18.9	20.6	22.3	24.0	25.7	27.3	28.9			$6 \times 3\frac{1}{2}$
*5 × 4					11.0	12.8	14.5	16.2	17.8	19.5	21.1	22.6	24.2					*5 × 4
$5 \times 3\frac{1}{2}$				8.7	10.4	12.0	13.6	15.2	16.8	18.3	19.8	21.3	22.7					$5 \times 3\frac{1}{2}$
5 × 3				8.2	9.8	11.3	12.8	14.2	15.7	17.1	18.5	19.9						5 × 3
*$4 \times 3\frac{1}{2}$				7.7	9.1	10.6	11.9	13.3	14.6	15.9	17.2	18.5						*$4 \times 3\frac{1}{2}$
4 × 3				7.1	8.5	9.8	11.1	12.3	13.6	14.8	16.0	17.1						4 × 3
$3\frac{1}{2} \times 3$				6.6	7.8	9.1	10.2	11.4	12.5	13.6	14.7	15.7						$3\frac{1}{2} \times 3$
$3\frac{1}{2} \times 2\frac{1}{2}$			4.9	6.1	7.2	8.3	9.4	10.4	11.4	12.4								$3\frac{1}{2} \times 2\frac{1}{2}$
$3 \times 2\frac{1}{2}$			4.5	5.5	6.6	7.6	8.5	9.5										$3 \times 2\frac{1}{2}$
3 × 2			4.0	5.0	5.9	6.8	7.7											3 × 2
$2\frac{1}{2} \times 2$		2.8	3.7	4.5	5.3	6.1	6.8											$2\frac{1}{2} \times 2$
SIZE	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1	$1\frac{1}{16}$	$1\frac{1}{8}$	SIZE

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TABLE XXXVII.

CARNEGIE ANGLES.

Areas in Square Inches.

ANGLES

Area in square inches.

SIZE	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	SIZE
8 × 8							7.75	8.68	9.61	10.53	11.44	12.34	13.23	14.12	15.00	15.87	16.73	8 × 8
6 × 6					4.36	5.06	5.75	6.43	7.11	7.78	8.44	9.09	9.74	10.37	11.00			6 × 6
*5 × 5					3.61	4.18	4.75	5.31	5.86	6.42	6.94	7.46	7.99	8.50	9.00			*5 × 5
4 × 4				2.40	2.86	3.31	3.75	4.18	4.61	5.03	5.44	5.84						4 × 4
3½ × 3½				2.09	2.48	2.87	3.25	3.62	3.98	4.34	4.69	5.03						3½ × 3½
3 × 3			1.44	1.78	2.11	2.43	2.75	3.06	3.36									3 × 3
*2¾ × 2¾			1.31	1.62	1.92	2.22	2.50											*2¾ × 2¾
2½ × 2½		0.90	1.19	1.47	1.73	2.00	2.25											2½ × 2½
*2¼ × 2¼		0.81	1.06	1.31	1.55	1.78	2.00											*2¼ × 2¼
2 × 2		0.72	0.94	1.15	1.36	1.56												2 × 2
1¾ × 1¾		0.62	0.81	1.00	1.17	1.30												1¾ × 1¾
1½ × 1½	0.36	0.53	0.69	0.84	0.99													1½ × 1½
1¼ × 1¼	0.30	0.43	0.56	0.69														1¼ × 1¼
1 × 1	0.24	0.34	0.44															1 × 1
SIZE	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	SIZE
*7 × 3½						4.40	5.00	5.59	6.17	6.75	7.31	7.87	8.42	8.97	9.50			*7 × 3½
6 × 4					3.61	4.18	4.75	5.31	5.86	6.41	6.94	7.47	7.99	8.50	9.00			6 × 4
6 × 3½					3.42	3.97	4.50	5.03	5.55	6.06	6.56	7.06	7.55	8.03	8.50			6 × 3½
*5 × 4					3.23	3.75	4.25	4.75	5.23	5.72	6.19	6.65	7.11					*5 × 4
5 × 3½				2.56	3.05	3.53	4.00	4.47	4.92	5.37	5.81	6.25	6.67					5 × 3½
5 × 3				2.40	2.86	3.31	3.75	4.18	4.61	5.03	5.44	5.84						5 × 3
*4 × 3½				2.25	2.67	3.09	3.50	3.90	4.30	4.68	5.06	5.43						*4 × 3½
4 × 3				2.09	2.48	2.87	3.25	3.62	3.98	4.34	4.69	5.03						4 × 3
3½ × 3				1.93	2.30	2.65	3.00	3.34	3.67	4.00	4.31	4.62						3½ × 3
3½ × 2½			1.44	1.78	2.11	2.43	2.75	3.06	3.36	3.65								3½ × 2½
3 × 2½			1.31	1.62	1.92	2.22	2.50	2.78										3 × 2½
3 × 2			1.19	1.47	1.73	2.00	2.25											3 × 2
2½ × 2		0.81	1.06	1.31	1.55	1.78	2.00											2½ × 2
SIZE	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	SIZE

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TABLE XXXVIII.

Upsets for Round and Square Bars.

ROUND BARS							SQUARE BARS						
ROUND		UPSET					UPSET					SQUARE	
Diam.	Area	Diam.	Length	Add	Area at Root	Excess Area	Excess Area	Area at Root	Add	Length	Diam.	Area	Diam.
Inches	Sq. In.	Inches	Inches	Inches	Sq. In.	%	%	Sq. In.	Inches	Inches	Inches	Sq. In.	Inches
5/8	0.307	7/8	4	4 1/2	0.420	36.8							5/8
3/4	0.442	1	4	3 7/8	0.550	24.4	20.6	0.694	3 1/2	4	1 1/8	0.563	3/4
7/8	0.601	1 1/4	4	5	0.891	48.3	16.3	0.891	4	4	1 1/4	0.766	7/8
1	0.785	1 3/8	4	4 3/8	1.057	34.7	29.5	1.295	4	4	1 3/8	1.000	1
1 1/8	0.994	1 1/2	4	3 7/8	1.295	30.3	19.7	1.515	4 1/2	4 1/2	1 1/2	1.266	1 1/8
1 1/4	1.227	1 5/8	4 1/2	3 7/8	1.515	23.5	31.1	2.049	4 1/4	4 1/2	1 7/8	1.563	1 1/4
1 3/8	1.485	1 3/4	4 1/2	3 1/2	1.744	17.4	21.7	2.302	4 1/2	5	2	1.891	1 3/8
1 1/2	1.767	2	5	4 5/8	2.302	30.3	34.0	3.023	4 1/4	5	2 1/8	2.250	1 1/2
1 5/8	2.074	2 1/8	5	4 1/4	2.651	27.8	29.6	3.410	4 5/8	5 1/2	2 3/8	2.641	1 5/8
1 3/4	2.405	2 1/4	5	4	3.023	25.7	21.3	3.716	4 1/4	5 1/2	2 1/2	3.063	1 3/4
1 7/8	2.761	2 3/8	5 1/2	4 1/2	3.410	23.9	31.4	4.619	5 1/2	6	2 5/8	3.516	1 7/8
2	3.142	2 1/2	5 1/2	3 7/8	3.716	18.3	27.7	5.107	4 3/4	6	2 7/8	4.000	2
2 1/8	3.547	2 5/8	5 1/2	3 5/8	4.155	17.1	20.2	5.430	4 5/8	6	3	4.516	2 1/8
2 1/4	3.976	2 7/8	6	4 5/8	5.107	28.5	28.6	6.510	5 1/2	6 1/2	3 1/4	5.063	2 1/4
2 3/8	4.430	3	6	4 3/8	5.430	22.6	33.8	7.548	6 1/2	7	3 1/2	5.641	2 3/8
2 1/2	4.909	3 1/8	6 1/2	4 5/8	5.957	21.3	30.7	8.170	6 1/4	8	3 5/8	6.250	2 1/2
2 5/8	5.412	3 1/4	6 1/2	4 1/4	6.510	20.3	35.0	9.305	6 3/4	8	3 7/8	6.891	2 5/8
2 3/4	5.940	3 3/8	7	4 1/4	7.088	19.3	32.1	9.994	6	8	4	7.563	2 3/4
2 7/8	6.492	3 1/2	8	5 1/2	8.170	25.9	37.0	11.329	8	9	4 1/4	8.266	2 7/8
3	7.069	3 3/4	8	5 1/4	8.641	22.2	41.7	12.753	7 1/2	9	4 1/2	9.000	3
3 1/8	7.670	3 7/8	8	5 1/2	9.305	21.3							3 1/8
3 1/4	8.296	4	8	4 7/8	9.994	30.7							3 1/4
3 1/2	9.621	4 1/8	9	5 1/2	11.329	17.7							3 1/2
3 3/4	11.045	4 1/4	9	4 3/4	12.753	15.5							3 3/4

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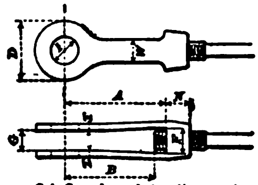
TABLE XXXIX.

CLEVISES. AMERICAN BRIDGE COMPANY STANDARDS.

All Dimensions in Inches.



DIAM. OF CLEVIS D	MAX. PIN P	CLEVIS FORK F	CLEVIS NUT N	CLEVIS WIDTH W	CLEVIS THICKNESS T	CLEVIS A	CLEVIS B	DIAM. OF CLEVIS D
3	1½	1½	1½	1½	½	6	5	3
4	2¼	1⅞	1⅞	1⅞	⅝	9	8	4
5	3	2¼	2¼	2¼	¾	9	8	5
6	3½	2¾	2¾	2¾	⅞	9	8	6
7	4	3¼	3¼	3¼	1	9	8	7

Table giving diameter of Clevis for given rod and pin.

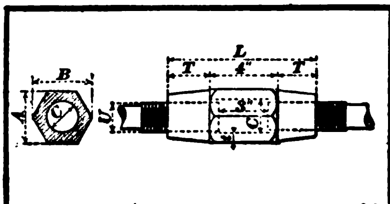
ROD ROUND	ROD SQUARE	ROD UPSET	PINS 1	1¼	1½	1¾	2	2¼	2½	2¾	3	3¼	3½	3¾	4	ROD UPSET	ROD SQUARE	ROD ROUND
¾	⅝	1	3	3	3											1	⅝	¾
	¾	1⅛	3	3	3	4	4	4								1⅛	¾	
⅞	⅞	1¼		4	4	4	4	4								1¼	⅞	⅞
1		1⅜		4	4	4	4	4								1⅜		1
1⅛	1	1½		4	4	4	4	4	5	5	5					1½	1	1⅛
1¼	1⅛	1⅝			4	4	5	5	5	5	5					1⅝	1⅛	1¼
1⅜		1¾			5	5	5	5	5	5	5					1¾		1⅜
	1¼	1⅞			5	5	5	5	5	5	5					1⅞	1¼	
1½	1⅜	2				5	5	5	5	5	6	6	6			2	1⅜	1½
1⅝		2⅛				5	5	5	5	6	6	6	6			2⅛		1⅝
1¾	1½	2¼					6	6	6	6	6	6	7	7	7	2¼	1½	1¾
1⅞	1⅝	2⅜					6	6	6	6	7	7	7	7	7	2⅜	1⅝	1⅞
2	1¾	2½						6	6	7	7	7	7	7	7	2½	1¾	2
2⅛		2⅝							7	7	7	7	7			2⅝		2⅛
	1⅞	2¾							7	7	7	7	7			2¾	1⅞	
2¼	2	2⅞								7	7					2⅞	2	2¼
ROUND	SQUARE	UPSET	1	1¼	1½	1¾	2	2¼	2½	2¾	3	3¼	3½	3¾	4	UPSET	SQUARE	ROUND
ROD			PINS													ROD		

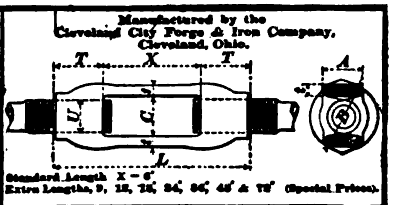
Clevises above and to right of heavy zigzag line, may be used with forks straight. Clevises below and to left of same line, should have forks closed in until pin is not overstrained.

TABLE XL.

Sleeve Nuts and Turnbuckles. American Bridge Company Standards.

All Dimensions in Inches.





Diam. of Screw U	Length of Thread T	Length of Nut L	Short Diam. A	Long Diam. B	Inside Diam. C	Thickness t	Weight in Lbs.	Weight in Lbs.	Standard Dimensions t	A	B	C	L	T	Diam. of Screw U
7/8	1 1/8	7	1 3/8	1 7/8	1 1/8	1/4	2 3/4	2 1/2	3/8	1 1/4	2 1/4	1 1/4	8 3/8	1 9/16	7/8
1	1 1/2	7	1 5/8	1 7/8	1 1/8	1/4	3	3 1/2	7/16	"	2 7/16	1 5/16	9	1 1/2	1
1 1/8	1 3/4	7 1/2	2	2 5/16	1 3/8	5/16	3 1/2	4	1/2	"	2 9/16	1 7/16	9 5/8	1 11/16	1 1/8
1 1/4	"	"	"	"	"	"	4	5 1/2	"	1 1/2	2 3/4	1 9/16	9 3/4	1 7/8	1 1/4
1 3/8	2	8	2 3/8	2 3/4	1 5/8	3/8	4 1/2	6	"	1 5/8	3 1/16	1 11/16	10 1/8	2 1/16	1 3/8
1 1/2	"	"	"	"	"	"	6 1/2	7	5/8	"	3 5/16	1 3/4	10 1/2	2 1/4	1 1/2
1 5/8	2 1/4	8 1/2	2 3/4	3 5/16	1 7/8	7/16	8	8 1/2	"	1 3/4	3 1/2	2	10 7/8	2 7/16	1 5/8
1 3/4	"	"	"	"	"	"	8 1/2	10	"	2	3 5/8	2 1/8	11 1/4	2 5/8	1 3/4
1 7/8	2 1/2	9	3 1/8	3 5/8	2 1/8	1/2	10	11 1/2	11/16	"	3 7/8	2 5/16	11 5/8	2 13/16	1 7/8
2	"	"	"	"	"	"	11	13	"	2 1/4	4 1/8	2 3/8	12	3	2
2 1/8	2 3/4	9 1/2	3 1/2	4 1/16	2 3/8	9/16	14	15	25/32	2 1/2	4 1/2	2 1/2	12 3/8	3 3/16	2 1/8
2 1/4	"	"	"	"	"	"	15	18	13/16	"	4 3/4	2 11/16	12 3/4	3 3/8	2 1/4
2 3/8	3	10	3 7/8	4 1/2	2 5/8	5/8	18	20	"	2 3/4	4 7/8	2 3/4	13 1/4	3 9/16	2 3/8
2 1/2	"	"	"	"	"	"	19	24	27/32	3	5 1/8	3 1/16	13 1/8	3 3/4	2 1/2
2 5/8	3 1/4	10 1/2	4 1/4	4 13/16	2 7/8	11/16	22	28	15/16	"	5 5/16	3 1/8	13 7/8	3 15/16	2 5/8
2 3/4	"	"	"	"	"	"	23	30	"	3 1/4	5 3/4	3 1/4	14 1/4	4 1/8	2 3/4
2 7/8	3 1/2	11	4 5/8	5 3/8	3 1/8	3/4	27	34	1 1/32	"	6 1/16	3 7/16	14 5/8	4 5/16	2 7/8
3	"	"	"	"	"	"	28	38	"	3 1/2	6 3/8	3 5/8	15	4 1/2	3
3 1/8	3 3/4	11 1/2	5	5 13/16	3 3/8	13/16	34								3 1/8
3 1/4	"	"	"	"	"	"	35	50	1 1/16	4	6 3/4	3 7/8	15 3/4	4 7/8	3 1/4
3 3/8	4	12	5 3/8	6 1/8	3 5/8	7/8	39								3 3/8
3 1/2	"	"	"	"	"	"	40	65	1 7/32	4	7 1/4	4 1/4	16 1/2	5 1/4	3 1/2
3 5/8	4 1/4	12 1/2	5 3/4	6 11/16	3 7/8	15/16	45								3 5/8
3 3/4	"	"	"	"	"	"	47		1 9/16	5	8 1/4	4 7/16	18	6	3 3/4
3 7/8	4 1/2	13	6 1/8	7 1/8	4 1/8	1	52								3 7/8
4	"	"	"	"	"	"	55		1 7/16	5	8 3/4	4 5/8	18	6	4
4 1/4	4 3/4	13 1/2	6 1/2	7 3/16	4 3/8	1 1/16	65								4 1/4
4 1/2	5	14	6 7/8	8	4 3/4	1 1/16	75								4 1/2

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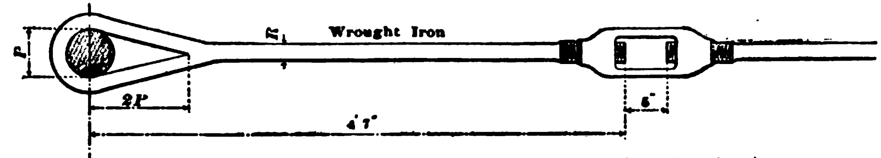
TABLE XLI.

LOOP BARS. AMERICAN BRIDGE COMPANY STANDARDS.

All Dimensions in Inches.



Length in inches beyond pin centre to form one eye equals 3.7 (P + R)

Diam. of Pins	¾	⅞	1	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2⅛	2¼	2⅜	2½	2⅝	2¾	2⅞	3	Diam. of Pins
	Diam. or Side of Bars																			
1	11½	12	12½																	1
1¼	12½	12⅞	13⅜	13⅞	14¼															1¼
1½	13⅜	13⅞	14⅜	14¾	15¼	15⅝	16⅛													1½
1¾	14¼	14¾	15¼	15⅝	16⅛	16⅝	17	17½	18											1¾
2	15¼	15⅝	16⅛	16⅝	17	17½	18	18⅜	18⅞	19⅜	19¾									2
2¼	16⅛	16⅝	17	17½	18	18⅜	18⅞	19⅜	19¾	20¼	20¾	21⅛	21⅝							2¼
2½	17	17½	18	18⅜	18⅞	19⅜	19¾	20¼	20¾	21⅛	21⅝	22⅛	22⅝	23	23½					2½
2¾	18	18⅜	18⅞	19⅜	19¾	20¼	20¾	21⅛	21⅝	22⅛	22⅝	23	23½	24	24⅜	24⅞	25⅜			2¾
3	18⅞	19⅜	19¾	20¼	20¾	21⅛	21⅝	22⅛	22⅝	23	23½	24	24⅜	24⅞	25⅜	25¾	26¼	26¾	27¼	3
3¼	19¾	20¼	20¾	21⅛	21⅝	22⅛	22⅝	23	23½	24	24⅜	24⅞	25⅜	25⅞	26¼	26¾	27¼	27⅝	28¼	3¼
3½	20¾	21⅛	21⅝	22⅛	22⅝	23	23½	24	24⅜	24⅞	25⅜	25⅞	26¼	26¾	27¼	27⅝	28⅛	28⅝	29	3½
3¾	21⅝	22⅛	22⅝	23	23½	24	24⅜	24⅞	25⅜	25⅞	26¼	26¾	27¼	27⅝	28⅛	28⅝	29	29½	30	3¾
4	22⅝	23	23½	24	24⅜	24⅞	25⅜	25⅞	26¼	26¾	27¼	27⅝	28⅛	28⅝	29	29½	30	30½	31	4
4¼	23½	24	24⅜	24⅞	25⅜	25⅞	26¼	26¾	27¼	27⅝	28⅛	28⅝	29	29½	30	30½	31	31⅜	31⅞	4¼
4½	24⅜	24⅞	25⅜	25⅞	26¼	26¾	27¼	27⅝	28⅛	28⅝	29	29½	30	30½	31	31⅜	31⅞	32¼	32¾	4½
4¾	25⅜	25⅞	26¼	26¾	27¼	27⅝	28⅛	28⅝	29	29½	30	30½	31	31⅜	31⅞	32¼	32¾	33¼	33¾	4¾
5	26¼	26¾	27¼	27⅝	28⅛	28⅝	29	29½	30	30½	31	31⅜	31⅞	32¼	32¾	33¼	33¾	34⅛	34⅝	5
5¼	27¼	27⅝	28⅛	28⅝	29	29½	30	30½	31	31⅜	31⅞	32¼	32¾	33¼	33¾	34⅛	34⅝	35	35½	5¼
5½	28⅛	28⅝	29	29½	30	30½	31	31⅜	31⅞	32¼	32¾	33¼	33¾	34⅛	34⅝	35	35½	36	36½	5½
5¾	29	29½	30	30½	31	31⅜	31⅞	32¼	32¾	33¼	33¾	34⅛	34⅝	35	35½	36	36½	37	37½	5¾
6	30	30½	31	31⅜	31⅞	32¼	32¾	33¼	33¾	34⅛	34⅝	35	35½	36	36½	37	37½	37⅞	38⅜	6

NOTE: Maximum shipping length should not exceed 35 feet.

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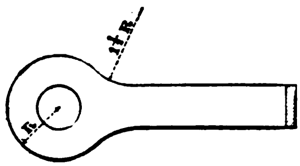
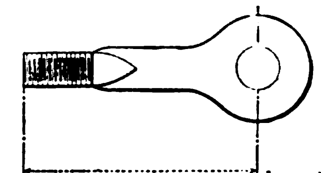
TABLE XLII.

EYE BARS. AMERICAN BRIDGE COMPANY STANDARDS.

Ordinary



Adjustable



Min. Length C. to end 6' 6" preferably 7' 0"

Width of Bar	Min. Thickness of Bar	Head: Diam.	Head: Max. Pin.	Head: Add'l Material for Head	Screw End: Add'l Material for Upset	Screw End: Diam.	Screw End: Length	Thickness of Bar	Width of Bar
Ins.	Ins.	Ins.	Ins.	Ft. & Ins.	Ft. & Ins.	Ins.	Ins.	Ins.	Ins.
2	½	4½	1¾	0-7½	0-7	2	5	13/16	2
	"	5½	2¾	1-0½					
2½	⅝	5½	2¼	0-9½	1-1	2¼	5	13/16 to 1 1/16	2½
	"	6½	3¼	1-1½					
3	¾	7	3	1-3	1-5	2½	5½	1 to 1 1/16	3
	"	8	4	1-6	1-5	2¾	6	1⅛ to 1¼	
4	¾	9½	4½	1-8	1-8	3	6	1 to 1⅛	4
	"	10½	5½	1-10	1-8	3¼	6½	1 3/16 to 1⅜	
5	¾	11½	5	1-9	1-9	3½	6½	1 to 1 1/16	5
	1	12½	6	2-1	1-9	3½	7	1⅛ to 1¼	
6	¾	13½	5½	1-11	1-11	3¾	8	1⅛ to 1 3/16	6
	1	14½	6½	2-2	1-11	4	8	1¼ to 1⅜	
7	⅞	16	6¾	2-3	2-3	4¼	9	1⅛ to 1 3/16	7
	15/16	17	7¾	2-8	2-3	4½	9	1¼ to 1½	
8	1	17	6½	2-3					8
	1 1/16	18	7½	2-6					
	1⅛	18½	8	2-10					
9	1⅛	19½	7¾	2-6					9
	"	21½	9½	3-1					
10	1¼	22	9	2-11					10
	"	23	10	3-3					
12									12

Note: Eye bars are hydraulic forged, and are guaranteed to develop the full strength of the bar, under conditions given in the above table, when tested to destruction.

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TABLE XLIII.

Maximum Allowable Bending Moments in Pins for Various Fiber Stresses.

PIN		MOMENTS IN INCH POUNDS FOR FIBRE STRESSES PER SQ. IN. OF				
DIAM. INCHES	AREA	15,000	18,000	20,000	22,000	25,000
1	0.785	1470	1770	1960	2160	2450
1¼	1.227	2880	3450	3830	4220	4790
1½	1.767	4970	5960	6630	7290	8280
1¾	2.405	7890	9470	10500	11570	13200
2	3.142	11800	14100	15700	17280	19600
2¼	3.976	16800	20100	22400	24600	28000
2½	4.909	23000	27600	30700	33700	38400
2¾	5.940	30600	36800	40800	44900	51000
3	7.069	39800	47700	53000	58300	66300
3¼	8.296	50600	60700	67400	74100	84300
3½	9.621	63100	75800	84200	92600	105200
3¾	11.045	77700	93200	103500	113900	129400
4	12.566	94200	113100	125700	138200	157100
4¼	14.186	113000	135700	150700	165800	188400
4½	15.904	134200	161000	178900	196800	223700
4¾	17.721	157800	189400	210400	231500	263000
5	19.635	184100	220900	245400	270000	306800
5¼	21.648	213100	255700	284100	312500	355200
5½	23.758	245000	294000	326700	359300	408300
5¾	25.967	280000	335900	373300	410600	466600
6	28.274	318100	381700	424100	466500	530200
6¼	30.680	359500	431400	479400	527300	599200
6½	33.183	404400	485300	539200	593100	674000
6¾	35.785	452900	543500	603900	664200	754800
7	38.485	505100	606100	673500	740800	841900
7¼	41.282	561200	673400	748200	823000	935300
7½	44.179	621300	745500	828400	911200	1035400
7¾	47.173	685500	822600	914000	1005300	1142500
8	50.265	754000	904800	1005300	1105800	1256600
8¼	53.456	826900	992300	1102500	1212800	1378200
8½	56.745	904400	1085200	1205800	1326400	1507300
8¾	60.132	986500	1183800	1315400	1446900	1644200

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TABLE XLIV.

TABLE FOR RIVET SPACING.

Spaces	Pitch in Inches															Spaces
	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2⅛	2¼	2⅜	2½	2⅝	2¾	2⅞	
1																1
2	- 2¼	- 2½	- 2¾	- 3	- 3¼	- 3½	- 3¾	- 4	- 4¼	- 4½	- 4¾	- 5	- 5¼	- 5½	- 5¾	2
3	- 3⅜	- 3¾	- 4⅛	- 4½	- 4⅞	- 5¼	- 5⅝	- 6	- 6⅜	- 6¾	- 7⅛	- 7½	- 7⅞	- 8¼	- 8⅝	3
4	- 4½	- 5	- 5½	- 6	- 6½	- 7	- 7½	- 8	- 8½	- 9	- 9½	-10	-10½	-11	-11½	4
5	- 5⅝	- 6¼	- 6⅞	- 7½	- 8⅛	- 8¾	- 9⅜	-10	-10⅝	-11¼	-11⅞	1- 0½	1- 1⅛	1- 1¾	1- 2⅜	5
6	- 6¾	- 7½	- 8¼	- 9	- 9¾	-10½	-11¼	1- 0	1- 0¾	1- 1½	1- 2¼	1- 3	1- 3¾	1- 4½	1- 5¼	6
7	- 7⅞	- 8¾	- 9⅝	-10½	-11⅜	1- 0¼	1- 1⅛	1- 2	1- 2⅞	1- 3¾	1- 4⅝	1- 5½	1- 6⅜	1- 7¼	1- 8⅛	7
8	- 9	-10	-11	1- 0	1- 1	1- 2	1- 3	1- 4	1- 5	1- 6	1- 7	1- 8	1- 9	1-10	1-11	8
9	-10⅛	-11¼	1- 0⅜	1- 1½	1- 2⅝	1- 3¾	1- 4⅞	1- 6	1- 7⅛	1- 8¼	1- 9⅜	1-10½	1-11⅝	2- 0¾	2- 1⅞	9
10	-11¼	1 0½	1- 1¾	1- 3	1- 4¼	1- 5½	1- 6¾	1- 8	1- 9¼	1-10½	1-11¾	2- 1	2- 2¼	2- 3½	2- 4¾	10
11	1- 0⅜	1- 1¾	1- 3⅛	1- 4½	1- 5⅞	1- 7¼	1- 8⅝	1-10	1-11⅜	2- 0¾	2- 2⅛	2- 3½	2- 4⅞	2- 6¼	2- 7⅝	11
12	1- 1½	1- 3	1- 4½	1- 6	1- 7½	1- 9	1-10½	2- 0	2- 1½	2- 3	2- 4½	2- 6	2- 7½	2- 9	2-10½	12
13	1- 2⅝	1- 4¼	1- 5⅞	1- 7½	1- 9⅛	1-10¾	2- 0⅜	2- 2	2- 3⅝	2- 5¼	2- 6⅞	2- 8½	2-10⅛	2-11¾	3- 1⅜	13
14	1- 3¾	1- 5½	1- 7¼	1- 9	1-10¾	2- 0½	2- 2¼	2- 4	2- 5¾	2- 7½	2- 9¼	2-11	3- 0¾	3- 2½	3- 4¼	14
15	1- 4⅞	1- 6¾	1- 8⅝	1-10½	2- 0⅜	2- 2¼	2- 4⅛	2- 6	2- 7⅞	2- 9¾	2-11⅝	3- 1½	3- 3⅜	3- 5¼	3- 7⅛	15
16	1- 6	1- 8	1-10	2- 0	2- 2	2- 4	2- 6	2- 8	2-10	3- 0	3- 2	3- 4	3- 6	3- 8	3-10	16
17	1- 7⅛	1- 9¼	1-11⅜	2- 1½	2- 3⅝	2- 5¾	2- 7⅞	2-10	3- 0⅛	3- 2¼	3- 4⅜	3- 6½	3- 8⅝	3-10¾	4- 0⅞	17
18	1- 8¼	1-10½	2- 0¾	2- 3	2- 5¼	2- 7½	2- 9¾	3- 0	3- 2¼	3- 4½	3- 6¾	3- 9	3-11¼	4- 1½	4- 3¾	18
19	1- 9⅜	1-11¾	2- 2⅛	2- 4½	2- 6⅞	2- 9¼	2-11⅝	3- 2	3- 4⅜	3- 6¾	3- 9⅛	3-11½	4- 1⅞	4- 4¼	4- 6⅝	19
20	1-10½	2- 1	2- 3½	2- 6	2- 8½	2-11	3- 1½	3- 4	3- 6½	3- 9	3-11½	4- 2	4- 4½	4- 7	4- 9½	20
21	1-11⅝	2- 2¼	2- 4⅞	2- 7½	2-10⅛	3- 0¾	3- 3⅜	3- 6	3- 8⅝	3-11¼	4- 1⅞	4- 4½	4- 7⅛	4- 9¾	5- 0⅜	21
22	2- 0¾	2- 3½	2- 6¼	2- 9	2-11¾	3- 2½	3- 5¼	3- 8	3-10¾	4- 1½	4- 4¼	4- 7	4- 9¾	5- 0½	5- 3¼	22
23	2- 1⅞	2- 4¾	2- 7⅝	2-10½	3- 1⅜	3- 4¼	3- 7⅛	3-10	4- 0⅞	4- 3¾	4- 6⅝	4- 9½	5- 0⅜	5- 3¼	5- 6⅛	23
24	2- 3	2- 6	2- 9	3- 0	3- 3	3- 6	3- 9	4- 0	4- 3	4- 6	4- 9	5- 0	5- 3	5- 6	5- 9	24
25	2- 4⅛	2- 7¼	2-10⅜	3- 1½	3- 4⅝	3- 7¾	3-10⅞	4- 2	4- 5⅛	4- 8¼	4-11⅜	5- 2½	5- 5⅝	5- 8¾	5-11⅞	25
26	2- 5¼	2- 8½	2-11¾	3- 3	3- 6¼	3- 9½	4- 0¾	4- 4	4- 7¼	4-10½	5- 1¾	5- 5	5- 8¼	5-11½	6- 2¾	26
27	2- 6⅜	2- 9¾	3- 1⅛	3- 4½	3- 7⅞	3-11¼	4- 2⅝	4- 6	4- 9⅜	5- 0¾	5- 4⅛	5- 7½	5-10⅞	6- 2¼	6- 5⅝	27
28	2- 7½	2-11	3- 2½	3- 6	3- 9½	4- 1	4- 4½	4- 8	4-11½	5- 3	5- 6½	5-10	6- 1½	6- 5	6- 8½	28
29	2- 8⅝	3- 0¼	3- 3⅞	3- 7½	3-11⅛	4- 2¾	4- 6⅜	4-10	5- 1⅝	5- 5¼	5- 8⅞	6- 0½	6- 4⅛	6- 7¾	6-11⅜	29
30	2- 9¾	3- 1½	3- 5¼	3- 9	4- 0¾	4- 4½	4- 8¼	5- 0	5- 3¾	5- 7½	5-11¼	6- 3	6- 6¾	6-10½	7- 2¼	30
Spaces	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2⅛	2¼	2⅜	2½	2⅝	2¾	2⅞	Spaces
	Pitch in Inches															

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TABLE XLV.

Table for Rivet Spacing.

Spaces	3	3⅛	3¼	3⅜	3½	3¾	4	4¼	4½	4¾	5	5¼	5½	5¾	6	Spaces
1																1
2	-6	-6¼	-6½	-6¾	-7	-7½	-8	-8½	-9	-9½	-10	-10½	-11	-11½	1-0	2
3	-9	-9⅜	-9¾	-10⅛	-10½	-11¼	1-0	1-0¾	1-1½	1-2¼	1-3	1-3¾	1-4½	1-5¼	1-6	3
4	1-0	1-0½	1-1	1-1½	1-2	1-3	1-4	1-5	1-6	1-7	1-8	1-9	1-10	1-11	2-0	4
5	1-3	1-3⅝	1-4¼	1-4⅞	1-5½	1-6¾	1-8	1-9¼	1-10½	1-11¾	2-1	2-2¼	2-3½	2-4¾	2-6	5
6	1-6	1-6¾	1-7½	1-8¼	1-9	1-10½	2-0	2-1½	2-3	2-4½	2-6	2-7½	2-9	2-10½	3-0	6
7	1-9	1-9⅞	1-10¾	1-11⅝	2-0½	2-2¼	2-4	2-5¾	2-7½	2-9¼	2-11	3-0¾	3-2½	3-4¼	3-6	7
8	2-0	2-1	2-2	2-3	2-4	2-6	2-8	2-10	3-0	3-2	3-4	3-6	3-8	3-10	4-0	8
9	2-3	2-4⅛	2-5¼	2-6⅜	2-7½	2-9¾	3-0	3-2¼	3-4½	3-6¾	3-9	3-11¼	4-1½	4-3¾	4-6	9
10	2-6	2-7¼	2-8½	2-9¾	2-11	3-1½	3-4	3-6½	3-9	3-11½	4-2	4-4½	4-7	4-9½	5-0	10
11	2-9	2-10⅜	2-11¾	3-1⅛	3-2½	3-5¼	3-8	3-10¾	4-1½	4-4¼	4-7	4-9¾	5-0½	5-3¼	5-6	11
12	3-0	3-1½	3-3	3-4½	3-6	3-9	4-0	4-3	4-6	4-9	5-0	5-3	5-6	5-9	6-0	12
13	3-3	3-4⅝	3-6¼	3-7⅞	3-9½	4-0¾	4-4	4-7¼	4-10½	5-1¾	5-5	5-8¼	5-11½	6-2¾	6-6	13
14	3-6	3-7¾	3-9½	3-11¼	4-1	4-4½	4-8	4-11½	5-3	5-6½	5-10	6-1½	6-5	6-8½	7-0	14
15	3-9	3-10⅞	4-0¾	4-2⅝	4-4½	4-8¼	5-0	5-3¾	5-7½	5-11¼	6-3	6-6¾	6-10½	7-2¼	7-6	15
16	4-0	4-2	4-4	4-6	4-8	5-0	5-4	5-8	6-0	6-4	6-8	7-0	7-4	7-8	8-0	16
17	4-3	4-5⅛	4-7¼	4-9⅜	4-11½	5-3¾	5-8	6-0¼	6-4½	6-8¾	7-1	7-5¼	7-9½	8-1¾	8-6	17
18	4-6	4-8¼	4-10½	5-0¾	5-3	5-7½	6-0	6-4½	6-9	7-1½	7-6	7-10½	8-3	8-7½	9-0	18
19	4-9	4-11⅜	5-1¾	5-4⅛	5-6½	5-11¼	6-4	6-8¾	7-1½	7-6¼	7-11	8-3¾	8-8½	9-1¼	9-6	19
20	5-0	5-2½	5-5	5-7½	5-10	6-3	6-8	7-1	7-6	7-11	8-4	8-9	9-2	9-7	10-0	20
21	5-3	5-5⅝	5-8¼	5-10⅞	6-1½	6-6¾	7-0	7-5¼	7-10½	8-3¾	8-9	9-2¼	9-7½	10-0¾	10-6	21
22	5-6	5-8¾	5-11½	6-2¼	6-5	6-10½	7-4	7-9½	8-3	8-8½	9-2	9-7½	10-1	10-6½	11-0	22
23	5-9	5-11⅞	6-2¾	6-5⅝	6-8½	7-2¼	7-8	8-1¾	8-7½	9-1¼	9-7	10-0¾	10-6½	11-0¼	11-6	23
24	6-0	6-3	6-6	6-9	7-0	7-6	8-0	8-6	9-0	9-6	10-0	10-6	11-0	11-6	12-0	24
25	6-3	6-6⅛	6-9¼	7-0⅜	7-3½	7-9¾	8-4	8-10¼	9-4½	9-10¾	10-5	10-11¼	11-5½	11-11¾	12-6	25
26	6-6	6-9¼	7-0½	7-3¾	7-7	8-1½	8-8	9-2½	9-9	10-3½	10-10	11-4½	11-11	12-5½	13-0	26
27	6-9	7-0⅜	7-3¾	7-7⅛	7-10½	8-5¼	9-0	9-6¾	10-1½	10-8¼	11-3	11-9¾	12-4½	12-11¼	13-6	27
28	7-0	7-3½	7-7	7-10½	8-2	8-9	9-4	9-11	10-6	11-1	11-8	12-3	12-10	13-5	14-0	28
29	7-3	7-6⅝	7-10¼	8-1⅞	8-5½	9-0¾	9-8	10-3¼	10-10½	11-5¾	12-1	12-8¼	13-3½	13-10¾	14-6	29
30	7-6	7-9¾	8-1½	8-5¼	8-9	9-4½	10-0	10-7½	11-3	11-10½	12-6	13-1½	13-9	14-4½	15-0	30
Spaces	3	3⅛	3¼	3⅜	3½	3¾	4	4¼	4½	4¾	5	5¼	5½	5¾	6	Spaces

PITCH IN INCHES

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TABLE XLVI.

TABLE OF AREAS IN SQUARE INCHES, TO BE DEDUCTED FROM RIVETED PLATES OR SHAPES TO OBTAIN NET AREAS.

THICKNESS PLATES IN INCHES.	SIZE OF HOLE. INCHES.													
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16
1/4	.06	.08	.09	.11	.13	.14	.16	.17	.19	.20	.22	.23	.25	.27
5/16	.08	.10	.12	.14	.16	.18	.20	.21	.23	.25	.27	.29	.31	.33
3/8	.09	.12	.14	.16	.19	.21	.23	.26	.28	.30	.33	.35	.38	.40
7/16	.11	.14	.16	.19	.22	.25	.27	.30	.33	.36	.38	.41	.44	.46
1/2	.13	.16	.19	.22	.25	.28	.31	.34	.38	.41	.44	.47	.50	.53
9/16	.14	.18	.21	.25	.28	.32	.35	.39	.42	.46	.49	.53	.56	.60
5/8	.16	.20	.23	.27	.31	.35	.39	.43	.47	.51	.55	.59	.63	.66
11/16	.17	.21	.26	.30	.34	.39	.43	.47	.52	.56	.60	.64	.69	.73
3/4	.19	.23	.28	.33	.38	.42	.47	.52	.56	.61	.66	.70	.75	.80
13/16	.20	.25	.30	.36	.41	.46	.51	.56	.61	.66	.71	.76	.81	.86
7/8	.22	.27	.33	.38	.44	.49	.55	.60	.66	.71	.77	.82	.88	.93
15/16	.23	.29	.35	.41	.47	.53	.59	.64	.70	.76	.82	.88	.94	1.00
1	.25	.31	.38	.44	.50	.56	.63	.69	.75	.81	.88	.94	1.00	1.06
1 1/16	.27	.33	.40	.46	.53	.60	.66	.73	.80	.86	.93	1.00	1.06	1.13
1 1/8	.28	.35	.42	.49	.56	.63	.70	.77	.84	.91	.98	1.05	1.13	1.20
1 3/16	.30	.37	.45	.52	.59	.67	.74	.82	.89	.96	1.04	1.11	1.19	1.26
1 1/4	.31	.39	.47	.55	.63	.70	.78	.86	.94	1.02	1.09	1.17	1.25	1.33
1 5/16	.33	.41	.49	.57	.66	.74	.82	.90	.98	1.07	1.15	1.23	1.31	1.39
1 3/8	.34	.43	.52	.60	.69	.77	.86	.95	1.03	1.12	1.20	1.29	1.38	1.46
1 7/16	.36	.45	.54	.63	.72	.81	.90	.99	1.08	1.17	1.26	1.35	1.44	1.53
1 1/2	.38	.47	.56	.66	.75	.84	.94	1.03	1.13	1.22	1.31	1.41	1.50	1.59
1 9/16	.39	.49	.59	.68	.78	.88	.98	1.07	1.17	1.27	1.37	1.46	1.56	1.66
1 5/8	.41	.51	.61	.71	.81	.91	1.02	1.12	1.22	1.32	1.42	1.52	1.63	1.73
1 11/16	.42	.53	.63	.74	.84	.95	1.05	1.16	1.27	1.37	1.47	1.58	1.69	1.79
1 3/4	.44	.55	.66	.77	.88	.98	1.09	1.20	1.31	1.42	1.53	1.64	1.75	1.86
1 13/16	.45	.57	.68	.79	.91	1.02	1.13	1.25	1.36	1.47	1.59	1.70	1.81	1.93
1 7/8	.47	.59	.70	.82	.94	1.05	1.17	1.29	1.41	1.52	1.64	1.76	1.88	1.99
1 15/16	.48	.61	.73	.85	.97	1.09	1.21	1.33	1.45	1.57	1.70	1.82	1.94	2.06
2	.50	.63	.75	.88	1.00	1.13	1.25	1.38	1.50	1.63	1.75	1.88	2.00	2.13

In calculating the net area add 1/8 inch to diameter of rivet before entering the table.

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CHAPTER XV.

Estimate of Weight and Cost of Mine Structures.

ESTIMATE OF WEIGHT.—The contract drawings for head frames and coal tipples are usually general drawings similar to those in Fig. 89 and Fig. 90, in which the main members are shown in position together with enough sketch details to enable the detailer to make the shop drawings. In making an estimate of weight from general drawings it is necessary that the engineer be familiar with the shop details, and that he know the necessary percentage of details to add to the main members to obtain the total shipping weight of the structure. The percentage of details will depend upon the make-up and design of the structure and the practice of the shop at which the structure is to be fabricated. Where the engineer has had experience with structures of the same kind it will be sufficiently accurate to add a percentage for details to the total weight of the main members. On account of the varying designs of head frames, coal tipples and mine structures it will usually be necessary to estimate the weight of each part of the structure separately. Before an engineer can make reliable estimates of steel frame structures from general drawings it will be necessary that he calculate the weights of numerous similar structures to obtain the necessary data as to per cent of details. The following data will be of assistance to the engineer in the absence of definite data for the particular case.

Details of steel riveted trusses made of angles will vary from 25 to 35 per cent of the main members. The details of columns made of two channels laced will vary from 35 to 50 per cent, depending upon the weight of the channels and the style of end connections. The weight of rivet heads in steel head frames will vary from 3 to 5 per cent of the total weight of the structure. In estimating the weight of corrugated steel add 25 per cent for end and side laps where two corrugation side lap and 6 in. end laps, and 15 per cent where one corrugation side lap and 4 in. end laps are specified.

Estimate of Weight of a Steel Head Frame.—A summary of a detailed estimate of the 75 ft. steel head frame built by the American

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Bridge Company at Tonopah, Nev., is given in Table XLVII. The details are 39.4 per cent of the weight of the main members. The rivet heads are 4.1 per cent of the weight of the structure.

TABLE XLVII.

ESTIMATE OF WEIGHT OF 75-FT. STEEL HEAD FRAME, TONOPAH-BELMONT MINING CO.

Member.	Weight in Lbs.		Total Weight, Lbs.	Details in Per Cent of Main Members.
	Main Members.	Details.		
Back braces	9,170	4,150	13,320	43
Front posts	3,590	2,790	6,380	77
Girders	5,446	1,250	6,696	23
Diaphragms	2,936	2,582	5,518	82
Channels	1,790	440	2,230	25
Angle struts	2,627	1,015	3,642	39
Channel struts	3,263	2,179	5,442	67
Stringers	1,466	613	2,079	43
Angle bracing	8,065	2,279	10,344	28
Steel girders	6,673	414	7,087	6
Total	45,026	17,712	62,738	39.4

TABLE XLVIII.

WEIGHTS AND PER CENTS OF DETAILS OF MILL BUILDINGS.

Steel Mill Buildings with Self Supporting Frames covered on Roof and Sides with one thickness of Corrugated Iron.								
Part of Structure	40'-0"x40'-0"x14'-0" 2 Trusses 40'-0" Pitch 1/4 End Framing Cor. Iron {Roof #24 Sides #24 2 Circular Ventilator		40'-0"x48'-0"x14'-0" 2 Trusses 40'-0" Pitch 1/4 End Framing Cor. Iron {Roof #24 Sides #24 2 Circular Ventilator		60'-0"x75'-0"x18'-0" 4 Trusses 60'-0" Pitch 1/5 End Framing Cor. Iron {Roof #22 Sides #24 3 Circular Ventilator		60'-0"x80'-0"x20'-0" 4 Trusses 60'-0" Pitch 1/5 End Framing Cor. Iron {Roof #22 Sides #24 Monitor Ventilators	
	Weight	Details in per cent of Main Members	Weight	Details in per cent of Main Members	Weight	Details in per cent of Main Members	Weight	Details in per cent of Main Members
	lbs	per cent	lbs.	per cent	lbs.	per cent	lbs	per cent
Trusses	2848	25	2848	25	13940	34.4	14020	24.6
4 L Columns	1428	70	1428	70	3476	52.5	4632	48.1
I Beam	1148	15	1148	15	4251	33.0	6056	14.1
L Columns	912	36	952	36	1470	14.0	1668	36.3
End Rafters	1036	17	1076	22	3314	11.6	1776	20.0
Eave Struts	900	0	1080	0	3117	33.2	2120	0.0
Lower Chord Bracing	930	22	1049	20	2763	9.7	3186	7.0
Rods	900	15	920	17	1737	6.2	2060	12.0
Purlins	2281	5	3516	7	6713	4.7	10595	0.0
Girts	3170	2	3252	2	9895	10.0	9590	0.0
Weight of Framework	15553	19	17269	20	50676	24.0	55703	13.0
Weight per Sq. Ft	9.8		9.0		11.2		11.7	
Corrugated Iron	5880		6892		17000		23626	
Total of Steel	21433		24161		67676		79329	
Weight per Sq. Ft.	13.4		12.6		15.1		16.5	

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Estimate of Weight of Steel Buildings.—The weights and per cents of details of four steel frame buildings are given in Table XLVIII. The first two buildings in the table are an engine and boiler house, respectively, built for the Winona Mine, Michigan, and the third and fourth are two transformer buildings built for the East Helena smelter, Montana. The plans of the fourth building are given in Figs. 160 to 163. A detailed estimate of a steel frame building is given in the author's "The Design of Steel Mill Buildings."

ESTIMATE OF COST.—The different designs for head frames, coal tipples, and other mine structures vary so much with local conditions and requirements that it is only possible to give data that may be used as a guide to the experienced estimator. The cost of steel frame structures may be divided into (1) cost of material, (2) cost of fabrication, (3) cost of erection, and (4) cost of transportation.

1. **Cost of Material.**—The prices of structural steel is quoted in cents per pound delivered f o b. on cars at the point at which the quotation is made. Current prices may be obtained from the Engineering News, Iron Age or other technical papers. The present prices (1911) f. o. b. Pittsburgh, Pa., are about as follows:

TABLE XLIX.

PRICES OF STRUCTURAL STEEL (1911) F. O. B PITTSBURG, PA., IN CENTS PER POUND.

Material,	Price in Cts. Per Lb.
I-beams, 18 in. and over	1.50
I-beams and channels, 3 in. to 15 in.	1.40
H-beams, over 8 in.	1.60
Angles, 3 in. to 6 in. inclusive	1.45
Angles, over 6 in.	1.50
Zees, 3 in. and over	1.45
Angles, channels, and zees, under 3 in.	1.40
Deck beams and bulb angles	1.70
Checkered and corrugated plates	1.75 to 1.90
Plates, structural, base	1.35
Plates, flange, base	1.50
Corrugated steel No. 22, painted	2.00
Corrugated steel No. 22, galvanized	2.40
Steel sheets Nos. 10 and 11, black	1.65
Steel sheets Nos. 10 and 11, galvanized	2.00
Steel sheets No. 22, black	1.85
Sheet steel No. 22, galvanized	2.40
Bar iron, base	1.40

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The prices above are net with the exception of those for plates and bars, which are subject to standard extras as follows:

Extras.—*Shapes, Plates and Bars:*

(Cutting to length)

Under 3 ft. to 2 ft., inclusive	0.25 cts. per lb.
Under 2 ft. to 1 ft., inclusive	0.50 cts. per lb.
Under 1 ft.	1.55 cts. per lb.

Extras—Plates (Card of January 7, 1902):

Base $\frac{1}{4}$ in. thick, 100 in. wide and under, rectangular (see sketches).

Weights—see Manufacturer's Standard Specifications, Carnegie or Cambria Hand-books.

	Per 100 Lbs.
Widths—100 in. to 110 in.	$ .05
110 in. to 115 in.	.10
115 in. to 120 in.	.15
120 in. to 125 in.	.25
125 in. to 130 in.	.50
Over 130 in.	1.00
Gages under $\frac{1}{4}$ in. to and including $\frac{3}{16}$ in.	.10
Gages under $\frac{3}{16}$ in to and including No. 8	.15
Gages under No. 8 to and including No 9	.25
Gages under No. 9 to and including No 10	.30
Gages under No. 10 to and including No. 12	.40
Complete circles	.20
Boiler and flange steel	.10
Marine and fire box	.20
Ordinary sketches	.10

(Except straight taper plates, varying not more than 4 in. in width at ends, narrowest end not less than 30 in., which can be supplied at base prices.)

TABLE L.

STANDARD CLASSIFICATION OF EXTRAS ON IRON AND STEEL BARS.*

Rounds and Squares.

Squares up to $4\frac{1}{4}$ inches only. Intermediate sizes take the next higher extra.

	Per 100 Lbs.
$\frac{3}{4}$ to 3 in.	Rates.
$\frac{5}{8}$ to $1\frac{1}{16}$ "	$0.10 extra.
$\frac{1}{2}$ to $\frac{9}{16}$ "	.20 "
$\frac{7}{16}$ "	.40 "
$\frac{3}{8}$ "	.50 "
$\frac{5}{16}$ "	.60 "
$\frac{1}{4}$ and $\frac{9}{32}$ "	.70 "

* Adopted August, 1902.

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Size		Per 100 Lbs
7/32 in		1.00 "
3/16 "		2.00 "
3 1/16 to 3 1/2 "		.15 "
3 9/16 to 4 "		.25 "
4 1/16 to 4 1/2 "		.30 "
4 9/16 to 5 "		.40 "
5 1/8 to 5 1/2 "		.50 "
5 5/8 to 6 "		.75 "
6 1/8 to 6 1/2 "		1.00 "
6 5/8 to 7 1/4 "		1.25 "

Flat Bars and Heavy Bands.

Width	Thickness	Per 100 Lbs.
1 to 6 in.	x 3/8 to 1 in.	Rates.
1 to 6 "	x 1/4 and 5/16 "	$0.20 extra.
11/16 to 15/16 "	x 3/8 to 3/4 "	.40 "
11/16 to 15/16 "	x 1/4 and 5/16 "	.50 "
9/16 and 5/8 "	x 3/8 to 1/2 "	.50 "
9/16 and 5/8 "	x 1/4 and 5/16 "	.70 "
1/2 "	x 3/8 and 7/16 "	.90 "
1/2 "	x 1/4 and 5/16 "	1.10 "
7/16 "	x 3/8 "	1.00 "
7/16 "	x 1/4 and 5/16 "	1.20 "
3/8 "	x 1/4 and 5/16 "	1.50 "
1 1/8 to 6 in.	x 1 1/16 to 1 3/16 in.	.10 "
1 1/8 to 6 "	x 1 1/4 to 1 1/2 "	.20 "
1 3/4 to 6 "	x 1 5/8 to 2 3/4 "	.30 "
3 1/8 to 6 "	x 3 to 4 "	.40 "

Light Bars and Bands.

Width	Thickness	Per 100 Lbs.
1 1/2 to 6 in.	x Nos. 7, 8, 9 and 3/16 in.	$0.40 extra.
1 1/2 to 6 in.	x Nos. 10, 11, 12 and 1/8 in.	.60 "
1 to 1 7/16 in.	x Nos 7, 8, 9 and 3/16 in.	.50 "
1 to 1 7/16 in	x Nos. 10, 11, 12 and 1/8 in.	.70 "
13/16 to 15/16 in.	x Nos. 7, 8, 9 and 3/16 in.	.70 "
13/16 and 15/16 in.	x Nos. 10, 11, 12 and 1/8 in.	.80 "
11/16 and 3/4 in.	x Nos. 7, 8, 9 and 3/16 in.	1.00 "
11/16 and 3/4 in.	x Nos. 10, 11, 12 and 1/8 in.	1.20 "
9/16 and 5/8 in.	x Nos. 7, 8, 9 and 3/16 in.	1.20 "
9/16 and 5/8 in.	x Nos. 10, 11, 12 and 1/8 in.	1.30 "
1/2	x Nos. 7, 8, 9 and 3/16 in.	1.30 "
1/2	x Nos. 10, 11, 12 and 1/8 in.	1.50 "
7/16	x Nos. 7, 8, 9 and 3/16 in.	1.80 "
7/16	x Nos. 10, 11, 12 and 1/8 in.	2.10 "
3/8	x Nos. 7, 8, 9 and 3/16 in.	1.90 "
3/8	x Nos. 10, 11, 12 and 1/8 in.	2.40 "

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Cost of Wire Rope.—The discount on wire rope is about 50 per cent from standard lists. The net prices of wire rope are (1911) about as follows, f. o. b. New Jersey mills.

Diameter, In.	Price per Foot.		
	Cast Steel.	Crucible Steel.	Plow Steel.
$2\frac{1}{2}$	$1.05	$1.24	$1.46
$2\frac{1}{4}$	.85	1.05	1.22
2	.57	.65	.76
$1\frac{3}{4}$	.44	.54	.64
$1\frac{1}{2}$	.32	.39	.45
$1\frac{1}{4}$	.23	.28	.32
1	.15	18	.21
$\frac{3}{4}$	.10	.11	.13
$\frac{5}{8}$	.07	.09	.10
$\frac{1}{2}$	.06	.07	.07

2. **COST OF FABRICATION.**—The cost of fabrication of structural steel work may be divided into (1) cost of drafting, (2) cost of mill details, (3) cost of shop labor.

Cost of Drafting.—Cost of drafting varies with the character of the work and the tonnage to be made from the details so that costs per ton may mean but little. Shop details for circular steel bins cost from $1.50 to $3.00 per ton; shop details for conical or hopper bins cost from $4.00 to $6.00 per ton, while details for rectangular bins cost from $2.00 to $4.00 per ton, including columns and bracing. The shop details for head frames and coal tipples will cost from $4.00 to $6.00 per ton, while the shop details for ordinary mill buildings will cost from $3.00 to $4.00 per ton.

For additional costs of detailing see the author's "The Design of Steel Mill Buildings" and "The Design of Walls, Bins and Grain Elevators."

Cost of Mill Details.—Bridge companies and rolling mills have agreed upon standard prices for cutting to size, riveting on connection angles and similar work. The American Bridge Company's card for cost of mill details is the one in most common use.

American Bridge Co.'s card of cost of mill details:

Mill Rates:

"*a*"—0.15 ct. per lb.

This covers:

Plain punching 1 size hole in web only,

Plain punching 1 size hole in one or both flanges.

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"*b*"—0.25 ct. per lb.

This covers plain punching one size hole in either web and one flange or web and both flanges. (The holes in the web and flange must be of the same size.)

"*c*"—0.30 ct. per lb.

This covers:

Punching of 2 size holes in the web only,

Punching of 2 size holes in one or both flanges.

"*d*"—0.35 ct. per lb.

This covers punching and assembling into girders. Coping, ordinary beveling, including riveting and bolting of standard connection angles (this class includes beams shipped with connection angles bolted).

"*e*"—0.40 ct. per lb.

This covers the punching of one size hole in the web and another size hole in the flanges.

"*f*"—0.15 ct. per lb.

This covers cutting to length with less variation than plus or minus $\frac{3}{8}$ ins.

"*r*"—0.50 ct. per lb.

This covers beams with cover plates, shelf angles and ordinary riveted beam work, unless they are charged under class "*d*."

If this work consists of bending or any unusual work, the beams should not be included in the beam classification but estimated the same as riveted work. On all material estimated for cost at mill rates, 10 cts. per 100 lbs. is to be allowed for painting and 5 cts. per 100 lbs. is to be allowed for drawings.

Fittings.—All fittings, whether loose or attached, such as angle connections, bolts, separators, tie rods, etc., whenever they are estimated on in connection with beams or channels, to be charged at 1.55 cts. per lb. over and above the base price. The extra charge for painting is to be added to the price for fittings also. The base price on which fittings are based is not the base price of the beams to which they are attached, but is in all cases the base price of beams 15 in. and under. The above rates will not include painting or oiling, which should be charged at the rate of 0.10 ct. per lb. for one coat, over and above the base price plus the extra specified above. For plain punched beams, where holes of more than two sizes are used, 0.15 ct. per lb. should be added for each additional size hole; for example, plain punched

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beams, where three size holes occur, would be indicated as "*e*" plus 0.15 ct.; four size holes as "*e*" plus 0.30 ct.; for example, a beam with $\frac{5}{8}$ in. and $\frac{3}{4}$ in. holes in the flanges and $\frac{5}{8}$ in. and $\frac{3}{4}$ in. holes in the web should be included in class "*e*."

Cutting to length can be combined with any of the other rates except "*d*" and would have to be indicated; for example, plain punching one size hole in either web and one flange, or web and both flanges, and cutting to length would be marked "*bf*," which would establish a total charge of 0.40 ct. per lb.

Note to class "*d*":

No extra charge can be rendered to this class for punching various size holes or cutting to length; in other words, if a beam is coped, or has connection angles riveted or bolted to it, it makes no difference how many size holes are punched in this beam—the extra will always be the same, namely, 0.35 ct.

Beams with shelf angles, short seat angles or cover plates are strictly not covered by card rates. They can be charged either under class "*d*," this rate covering only the beam proper, in which case all other material ought to be rated as fittings with the charge of 1.55 cts. per lb. over and above the base price, or they can be classified under a special shop rate, "*r*,"—0.50 ct. per lb. This rate applies to all material forming the piece. It is the intention to charge whatever figures are the lowest, in order to give the customer the benefit of the doubt. In preparing the estimate, beam material should be marked with the letter "*b*" and to this should be added the letter giving the classification, thus: A beam punched with one size hole in one or both flanges will be marked "*b a*," etc.

Mill Orders.—In mill orders the following items should be borne in mind. Where beams butt at each end against some other member, order the beams $\frac{1}{2}$ inch shorter than the figured lengths; this will allow a clearance of $\frac{1}{4}$ inch if all beams come $\frac{3}{8}$ of an inch too long. Where beams are to be built into the wall, order them in full lengths, making no allowance for clearance. Order small plates in multiple lengths. Irregular plates on which there will be considerable waste should be ordered cut to templet. Mills will not make reentrant cuts in plates. Allow $\frac{1}{4}$ of an inch for each milling for members that have to be faced. Order web plates for girders $\frac{1}{4}$ to $\frac{1}{2}$ inch narrower than the distance back to back of angles. Order as nearly as possible every thing cut

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to required length, except where there is liable to be changes made, in which case order long lengths.

It is often possible to reduce the cost of mill details by having the mills do only part of the work, the rest being done in the field, or by sending out from the shop to be riveted on in the field connection angles and other small details that would cause the work to take a very much higher price Standard connections should be used wherever possible, and special work should be avoided.

The classification of iron and steel bars is given in Table L The full extra charges for sizes other than those taking the base rate are seldom enforced; one half card extras being very common.

In estimating the cost of plain material in a finished structure the shipping weight from the structural shop is wanted. The cost of material f. o. b. the shop must therefore include the cost of waste, paint material, and the freight from the mill to the shop. The waste is variable but as an average may be taken at 4 per cent. Paint material may be taken as one dollar per ton. The cost of plain material at the shop would be

	Cts.
Average cost per pound f. o. b. mill say	1.75
Add 4 per cent for waste	.07
Add $1.00 per ton for paint material	.05
Add freight from mill to shop (Pittsburg to St. Louis)	.225
Total cost per pound f. o. b. shop	2.095

To obtain the average cost of steel per pound multiply the pound price of each kind of material by the percentage that this kind of material is of the whole weight, the sum of the products will be the average pound price.

SHOP COSTS.—The following estimated shop costs include the cost of detailing and shop labor. The costs are based upon the average charge of 40 cents per hour for all labor.

Columns.—In lots of at least six, the shop cost of columns is about as follows: Columns made of two channels and two plates, or two channels laced cost about 0.80 to 0.70 cent per lb., for columns weighing from 600 to 1,000 lbs. each; columns made of 4 angles laced cost from 0.80 to 1.10 cents per lb.; columns made of two channels and one I-beam, or three channels cost from 0.65 to 0.90 cent per lb.; columns made of single I-beams, or single angles cost about 0.50 cent per lb.; and Z-bar columns cost from 0.70 to 0.90 cent per lb.

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Plain cast columns cost from 1.50 to 0.75 cents per lb., for columns weighing from 500 to 2,500 lbs., in lots of at least six.

Roof Trusses.—In lots of at least six, the shop cost of ordinary riveted roof trusses in which the ends of the members are cut off at right angles is about as follows: Trusses weighing 1,000 lbs. each, 1.15 to 1.25 cents per lb.; trusses weighing 1,500 lbs. each, 0.90 to 1.00 cent per lb.; trusses weighing 2,500 lbs. each, 0.75 to 0.85 cent per lb.; and trusses weighing 3,500 to 7,500 lbs 0.60 to 0.75 cent per lb. Pin connected trusses cost from 0.10 to 0.20 cent per lb. more than riveted trusses.

Eave Struts.—Ordinary eave struts made of 4 angles laced, whose length does not exceed 20 to 30 feet, cost for shop work from 0.80 to 1.00 cent per lb.

Plate Girders.—The shop work on plate girders for crane girders and floors will cost from 0.60 to 1.25 cents per lb., depending upon the weight, details and number made at one time.

Eye-Bars.—The shop cost of eye-bars varies with the size and length of the bars and the number made alike. The following costs are a fair average: Average shop cost of bars 3 in. and less in width and $\frac{3}{4}$ in. and less in thickness, is from 1.20 to 1.85 cents per lb., depending on length and size. A good order of bars running from $2\frac{1}{2}$ in. $\times \frac{3}{4}$ in. to 3 in. $\times \frac{3}{4}$ in., and from 16 to 30 ft. long, with few variations in size, will cost about 1.20 cents per lb. Large bars in long lengths ordered in large quantities can be fabricated at from 0.55 to 0.75 cent per lb.

To get the total cost of eye-bars the cost of bar steel must be added to the shop cost.

Bins.—Shop costs for circular and rectangular bins are given in Table LI, while shop costs for bin bottoms are given in Table LII.

TABLE LI.

SHOP COST OF CIRCULAR AND RECTANGULAR BINS NOT INCLUDING HOPPERS OR BOTTOMS.

Thickness of Metal, In	Shop Cost in Cents per Lb.
$\frac{1}{4}$	0.80
$\frac{5}{16}$	0.75
$\frac{3}{8}$	0.70
$\frac{1}{2}$	0.65

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TABLE LII.

SHOP COST OF BOTTOMS FOR CIRCULAR AND RECTANGULAR BINS.

Thickness of Material, In.	Flat Bottom, Cents per Lb.	Spherical Bottom, Cents per Lb.	Conical Bottom, Cents per Lb.	Hopper Bottom, Cents per Lb.
$\frac{1}{4}$	1.50	4.00	3.50	2.50
$\frac{5}{16}$	1 45	4.15	3.00	2.40
$\frac{3}{8}$	1.40	4.40	2.75	2.25
$\frac{1}{2}$	1.25	4 50	2.50	2 00

Steel Head Works.—Shop costs for ten steel frames are given in Table LIII and vary from $21.80 to $48.80 per ton. The shop costs of four steel coal tipples are given in Table LIV. The shop costs of steel head frames and coal tipples are high, due to the large percentage of details and skew connections.

3. **Cost of Erection.**—In estimating the cost of erection of structural steel work it is best to divide the cost into (*a*) cost of placing and bolting the steel, and (*b*) cost of riveting. The cost will be based on labor at $3.00 per day of 8 hours.

(*a*) *Cost of Placing and Bolting.*—The cost of placing and bolting up steel bins may be estimated at from $10.00 to $15.00 per ton. The cost of placing and bolting up head frames may be estimated at from $12.00 to $18.00 per ton.

(*b*) *Cost of Riveting.*—It will cost from 6 to 10 cents per rivet to drive $\frac{5}{8}$ or $\frac{3}{4}$ in. rivets by hand in structural framework where a few rivets are found in one place. A fair average is 7 cents per rivet. The same size rivets can be driven in tank work for from 4 to 7 cents per rivet, with 5 cents per rivet as a fair average.

The cost of riveting by hand is distributed about as follows:

3 men, 2 driving and 1 bucking up, at $3.50 per day of 8 hours..	$10.50
1 rivet heater at $3.00 per day of 8 hours....................	3.00
Coal, tools, superintendence..............................	1.50
Total per day....................................	$15.00

On structural work a fair day's work driving $\frac{3}{4}$ in. or $\frac{5}{8}$ in. rivets will be from 150 to 250, depending upon the amount of scaffolding required. This makes the total cost from 6 to 10 cents per rivet.

On bin work when the rivets are close together and little staging is required the gang above will drive from 200 to 400 rivets per day. This makes the total cost from about 4 to 7 cents per rivet.

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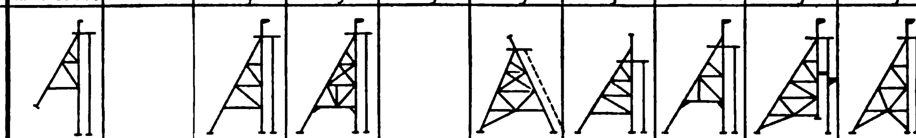
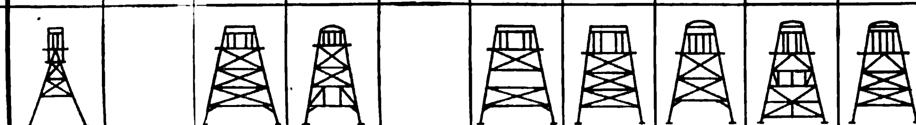
TABLE LIII.

Data on Steel Head Frames.

Mine	Wellman & Seavers St. Lawrence	B. & M.'s West Colusa	Anaconda	High Ore	Diamond	Steward	Grand Central	B. & M.'s Mt. View	Basin	Second Speculator
Year Designed	1897	1897	5-24-'98	Sept. 1898	Sept. 1898	Sept. 1898	11-16-'98	3-1-'99	4-15-'99	5-9-'99
Order Number	Estimate	Estimate	787	1200	1216	1257	S.L. #62	Estimate	551	648
Description—Height-Floor to ¢ Sheave	97'-0"	70'-0"	58'-8"	100'-0"	100'-0"	Vert. 55'-0"	50'-0"	80'-0"	70'-0"	50'-0"
Dimensions bet. Bases	48'×39' F-9B		34'×32'	61'×38'	61'×38'	47'×26'	40'×20'	65'×40' F-32B	45'×35'	30'×23'
Platform Floor	3/8" Check. Pls.	Plank Floor	5/16" Check Pls.	5/16" Check. Pls.	5/16" Check. Pls.	Plank Floor	Plank Floor	3/8" Check. Pls.	Plank Floor	5/16" Check. Pls.
Stairs	Check. Treads	Wood Stair	Cast Treads	Check. Treads	Check. Treads	Wood Ladder	Wood Treads	Cast Treads	Wood Treads	Cast Treads
Guides	2-10" Is	Wood	2-10" Is	12" & 10" Is	12" & 10" Is	Wood 10"×14"	7" [	2-10" Is	2-7" Is latt.	7" & 10" Is
Bins or Chutes	None	None	None	None	Skip Chutes 26000	None	None	None	None	None
Front Legs	4 Zs 6"×7/16"	2-15" B	4 Zs 6"×3/8"	Cov. 20"×7/16"-Webs 18"×7/16"-4 Ls 3 1/2"×3 1/2"×[illegible]		2-9" B	2-8" B	2 Pls. 20"×3/8"-2-15" B	2-9" B	2-7" B
Back Legs	1 Pl. 15 1/2"×7/16"	4 Ls 4"×4"×9/16"	1 Pl. 8"×3/8"	Batt.-Lacing 4"×3/8"		2-9" B	2-8" B	2 Pls. 20"×3/8"-2-15" B	2-10" B	2-8" B
Bracing	2-12" B & 9"	2-10" & 12" B	2-9" B-5"×3"×5/16" Ls	2-15" or 17" B	2-15" or 12" B	2-6, 7" & 8" B	4 Ls 2 1/2"×2"×1/4"	2 B 9, 10, 12 & 15"	2-7" & 6" B	2-5" B
Cross Girders	2-12"×30" B & Pls.	15" Is	2-15" B	Cov. 27"×3/8"-Webs 36"×1/2"-4 Ls 6"×4"×1/2"		2-12" B	2-15" B	4 Ls 4"×3"×5/16"-2 Pls. 36"×[illegible] 2 Pls. 12"×[illegible]	Front 2-12" B-Back 2-10" B	F. 2-8" B-B. 2-10" B
Bases & Anchors	Not Included	Not Incl.	Not Incl.	Not Incl.	Not Incl.	Included	Not Incl.	Included	Included	Included
Weight—	117000	56000	74700	292000	26000+292000	45000	133600	183000	79000	42200
Cost—Plain Material—Minneapolis—per lb.	1.50		1.52	1.41	1.41	1.53		1.65 (Acct. Pls.)	1.88	2.09
Shop Labor	1.10		1.56	1.09	1.09	2.08	1.81	1.55	1.64	2.44
Freight	They pay frt.		They pay frt.	They pay frt.	They pay frt.	0.80		0.80	0.80	0.80
Erection	0.75		0.61	0.56	0.56	0.76		They erect	0.56	0.60
Total Cost			$2755	$8940	$9740	$2327			$3857	$2500
Contract Price	$3920		3250	9487	10342	2200		$7320	4500	3050
	Wellman & Seavers	Jones & L's.	G.H. Mfg. Co.	G.H. Mfg. Co.	G.H. Mfg. Co.	G.H. Mfg. Co.	G.H. Mfg. Co.	Milwaukee B. Co.	G.H. Mfg. Co.	G.H. Mfg. Co.





Rivets can be driven by power riveters for one half to three fourths the above, not counting the cost of installation.

Soft iron rivets ½ in. and under can be driven cold for about one half what the rivet can be driven hot, or even less.

Small steel frame buildings like those given in Table XLVIII will cost about \$10.00 per ton for the erection of the steel framework, if trusses are riveted and all other connections are bolted. The cost of laying corrugated steel is about \$0.75 per square when laid on plank sheathing, \$1.25 per square when laid directly on the purlins, and \$2.00 per square when laid with anti-condensation lining. The erection of corrugated steel siding costs from \$0.75 to \$1.00 per square. The cost of erecting heavy machine shops, all material riveted and including the cost of painting but not the cost of the paint, is about \$8.50 to \$9.00 per ton. Small buildings in which all connections are bolted may be erected for from \$5.00 to \$6.00 per ton. The cost of erecting the East Helena transformer building (next to the last building in Table XLVIII) was \$12.80 per ton, including the erection of the corrugated steel and transportation of the men. The cost of erecting the Carbon Tipple was \$8.80 per ton, including corrugated steel. The cost of erection of the Basin & Bay State Smelter was \$8.20 per ton, including the hoppers and corrugated steel.

The shop costs, costs of erection, weight and other data for ten steel head frames are given in Table LIII. With the exception of the Grand Central all the head frames were built in the Butte district. The actual shipping weights of fifteen steel head frames are given in Table XIX, and of eight coal tipples in Table XX. The shop and erection costs of three steel coal tipples and a steel coal washer are given in Table LIV.

TABLE LIV.

Cost of Steel Coal Tipples and a Coal Washer.

Designed and Erected by The Gillette-Herzog Mfg. Co. in Montana.

Name.	Date.	Size.	Weight. Lbs.		Cost per Ton.	
			Structural Steel.	Corr. Steel.	Shop.	Erection.
Carbon Coal Co. Tipple..	1898	41′ × 150′ × 44′	355,400	16,800	\$20.20	\$9.00
R. F. Co. Tipple.........	1899	31′ × 124′ × 40′	171,200	31,300	\$18.60	\$8.80
Gebo Tipple................	1898	27′ × 98′ × 55′	117,200	10,300	\$13.40	\$6.60
Belt Coal Washer..........	1898	44′ × 72′ × 50½′	204,900	12,400	\$16.00	\$8.00

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Cost of Painting.—The amount of materials to make a gallon of paint and the surface covered by one gallon are given in Table LV.

TABLE LV.

AVERAGE SURFACE COVERED PER GALLON OF PAINT.

Paint	Volume of Oil.	Pounds of Pigment.	Volume and Weight of Paint.	Square Feet.	
				1 Coat.	2 Coats.
			Gals Lbs		
Iron oxide (powdered)......	1 gal.	8.00	1.2 = 16.00	600	350
Iron oxide (ground in oil)..	1 "	24.75	2.6 = 32.75	630	375
Read lead (powdered)......	1 "	22.40	1.4 = 30.40	630	375
White lead (ground in oil)..	1 "	25.00	1.7 = 33.00	500	300
Graphite (ground in oil).....	1 "	12.50	2.0 = 20.50	630	350
Black asphalt..................	1 " (turp.)	17.25	4.0 = 30.00	515	310
Linseed oil (no pigment) ...	1 "	.....	....	875	......

Light structural work will average about 250 square feet and heavy structural work about 150 square feet of surface per net ton of metal, while No. 20 corrugated steel has 2,400 square feet of surface.

It is the common practice to estimate ½ gallon of paint for the first coat and ⅜ gallon for the second coat per ton of structural steel, for average conditions.

The price of paint materials in small quantities in Chicago are (1911) about as follows: Linseed oil, 50 to 60 cents per gal.; iron oxide, 1 to 2 cents per lb.; red lead, 7 to 8 cents per lb.; white lead, 6 to 7 cents per lb.; graphite, 6 to 10 cents per lb.

A good painter should paint 1,200 to 1,500 square feet of plate surface or corrugated steel or 300 to 500 square feet of structural steel work in a day of 8 hours; the amount covered depending upon the amount of staging and the paint. A thick red lead mixed with 30 lbs. of lead to the gallon of oil will take fully twice as long to apply as a graphite paint or linseed oil.

For additional information on paints and painting see Chapter IX, also see "The Design of Steel Mill Buildings," Chapter XXVII.

Estimate of Cost of a Steel Head Frame.—The estimated cost of the 75 ft. steel head frame built by the American Bridge Company for the Tonopah-Belmont Mining Co., Tonopah, Nev., will be calculated, based on present (1911) prices. The estimate of the weight of the head frame is given in Table XLVII.

Cost of Plain Material.—The cost of plain material f. o. b. Pittsburgh will be as follows:

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Channels and beams	19,500 lbs. @ 1.40 =	$273.00
Angles 3 in. to 6 in.	17,543 lbs. @ 1.45 =	254.57
Angles under 3 in. and bars	10,065 lbs. @ 1.40 =	140.91
Plates	15,630 lbs. @ 1.35 =	211.00
	62,738	$879.48

Average cost of plain material = 879.48/62.738 = 1.40 cents per pound.

The cost of structural steel per pound f. o. b. shop will be as follows:

	Cts.
Average cost per pound f o. b. mill	1.40
Add 4 per cent for waste	0.056
Add $1.00 pe1 ton for paint material	0.05
Add freight from mill to shop (Pittsburg to Chicago)	0.165
Total cost per pound f. o. b. shop	1.671

The shop cost will be taken at 1.50 cts. per pound and the cost of erection at $30.00 per ton, not including field painting. Structural steel in car load lots takes fifth-class rate.

ESTIMATE OF COST OF 75-FT. STEEL HEAD FRAME.

Structural steel f. o. b. shop 62,738 lbs. @ 3.171 c. =	$1,990.42
Freight shop to site, 32 tons @ $20.00 (estimated) =	640.00
Erection, 31.5 tons @ $30.00 =	945.00
Paint for field coat, 30 gallons @ $1.50 =	45.00
Labor painting, 10 days @ $4.00 =	40.00
	$3,660.42
Profit 15 per cent	$ 549
Total cost erected	$4,209.42

Miscellaneous Costs.—For costs of roof coverings, floors, windows, doors, corrugated steel, and many other details of steel frame buildings, see the author's "The Design of Steel Mill Buildings." Additional data on bins of various types are given in the author's "The Design of Walls, Bins and Grain Elevators."

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APPENDIX I.

GENERAL SPECIFICATIONS FOR STEEL MINE STRUCTURES.

BY

MILO S. KETCHUM,

M. Am. Soc. C. E.

PART I. STEEL FRAME BUILDINGS.

PART II. STEEL HEAD FRAMES AND COAL TIPPLES, WASHERS AND BREAKERS.

1912.

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PART I. STEEL FRAME BUILDINGS.

GENERAL DESCRIPTION.

1. **Height of Building.**—The height of the building shall be the distance from the top of the masonry to the under side of the bottom chord of the truss.

2. **Dimensions of Building.**—The width and length of the building shall be the extreme distance out to out of framing or sheathing.

3. **Length of Span.**—The length of trusses and girders in calculating stresses shall be considered as the distance from center to center of end bearings when supported, and from end to end when fastened between columns by connection angles.

4. **Pitch of Roof.**—The pitch of roof for corrugated steel shall preferably be not less than $\frac{1}{4}$ (6″ in 12″), and in no case less than $\frac{1}{6}$. For a pitch less than $\frac{1}{6}$ some other covering than corrugated steel shall be used.

5. **Spacing of Trusses.**—Trusses shall be spaced so that simple shapes may be used for purlins. The spacing should be about 16 feet for spans of, say, 50 feet and about 20 to 22 feet for spans of, say, 100 feet. For longer spans than 100 feet the purlins may be trussed and the spacing may be increased.

6 **Spacing of Purlins.**—Purlins shall be spaced not to exceed 4′ 9″ where corrugated steel is used, and shall be placed at panel points of the trusses.

7. **Form of Trusses.**—The trusses shall preferably be of the Fink type with panels so subdivided that panel points will come under the purlins. If it is not practicable to place the purlins at panel points, the upper chords of the trusses shall be designed to take both the flexural and direct stresses. Trusses shall preferably be riveted trusses.

8. **Bracing.**—Bracing in the plane of the lower chords shall be stiff; bracing in the planes of the top chords, the sides and the ends may be made adjustable.

9. **Proposals.**—Contractors in submitting proposals shall furnish complete stress sheets, general plans of the proposed structures giving

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sizes of material, and such detail plans as will clearly show the dimensions of the parts, modes of construction and sectional areas.

10. **Detail Plans.**—The successful contractor shall furnish all working drawings required by the engineer free of cost. Working drawings will, as far as possible, be made on standard size sheets 24″ × 36″ out to out, 22″ × 34″ inside the inner border lines.

11. **Approval of Plans.**—No work shall be commenced or materials ordered until the working drawings are approved in writing by the engineer. The contractor shall be responsible for dimensions and details on the working plans, and the approval of the detail plans by the engineer will not relieve the contractor of this responsibility.

Loads.

12. The trusses shall be designed to carry the following loads:

13. **DEAD LOADS. Weight of Trusses.**—The weight of trusses per square foot of horizontal projection, up to 150 feet span shall be calculated by the formula

$$W = \frac{P}{45}\left(1 + \frac{L}{5\sqrt{A}}\right)$$

where W = weight of trusses per square foot of horizontal projection;
P = capacity of truss in pounds per square foot of horizontal projection;
L = span of the truss in feet;
A = distance between trusses in feet.

14. **Weight of Covering. Corrugated Steel.**—The weight of corrugated steel shall be taken from Table I.

TABLE I.
Weight of Flat and Corrugated Steel Sheets with 2½-inch Corrugations.

Gage No.	Thickness in Inches.	Weight per Square (100 Sq. Ft.).			
		Flat Sheets.		Corrugated Sheets.	
		Black.	Galvanized.	Black Painted.	Galvanized.
16	.0625	250	266	275	291
18	0500	200	216	220	236
20	.0375	150	166	165	182
22	.0313	125	141	138	154
24	.0250	100	116	111	127
26	.0188	75	91	84	99
28	.0156	63	79	69	86

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When two corrugations side lap and six inches end lap are used, add 25 per cent to the above weights; when one corrugation side lap and four inches end lap are used, add 15 per cent to the above weights to obtain weight of corrugated steel laid. For paint add 2 pounds per square. The weight of covering shall be reduced to weight per square foot of horizontal projection before combining with the weight of trusses.

15. **Slate.**—Slate laid with 3 inch lap shall be taken at a weight of $7\frac{1}{2}$ pounds per square foot of inclined roof surface for $\frac{3}{16}''$ slate $6'' \times 12''$, and $6\frac{1}{2}$ pounds per square foot of inclined roof surface for $\frac{3}{16}''$ slate $12'' \times 24''$, and proportionately for other sizes.

16. **Tile.**—Terra-cotta tile roofing weighs about 6 pounds per square foot for tile 1 inch thick; the actual weight of tile and other roof coverings not named shall be used.

17. **Sheathing and Purlins.**—Sheathing of dry pine lumber shall be assumed to weigh 3 pounds per foot and dry oak purlins 4 pounds per foot board measure.

18. **Miscellaneous Loads.**—The exact weight of sheathing, purlins, bracing, ventilators, cranes, etc., shall be calculated.

19. **SNOW LOADS.**—Snow loads shall be taken from the diagram in Fig. 1.

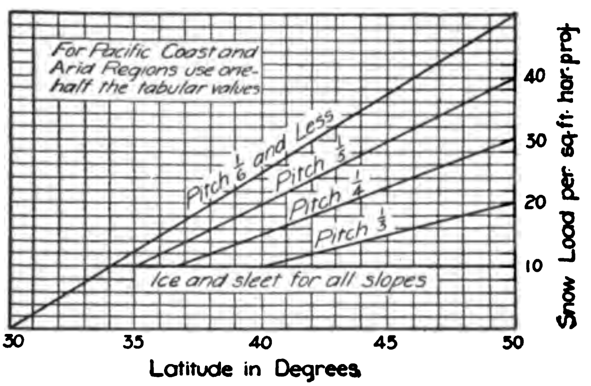

FIG. 1. SNOW LOAD ON ROOFS FOR DIFFERENT LATITUDES, IN LBS. PER SQUARE FOOT.

20. **WIND LOADS.**—The normal wind pressure on trusses shall be computed by Duchemin's formula, Fig. 2, with $P = 30$ pounds per square foot, except for buildings in exposed locations, where $P = 40$ pounds per square foot shall be used.

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21. The sides and ends of buildings shall be computed for a normal wind load of 20 pounds per square foot of exposed surface for buildings 30 feet and less to the eaves; 30 pounds per square foot of exposed surface for buildings 60 feet to the eaves, and in proportion for intermediate heights.

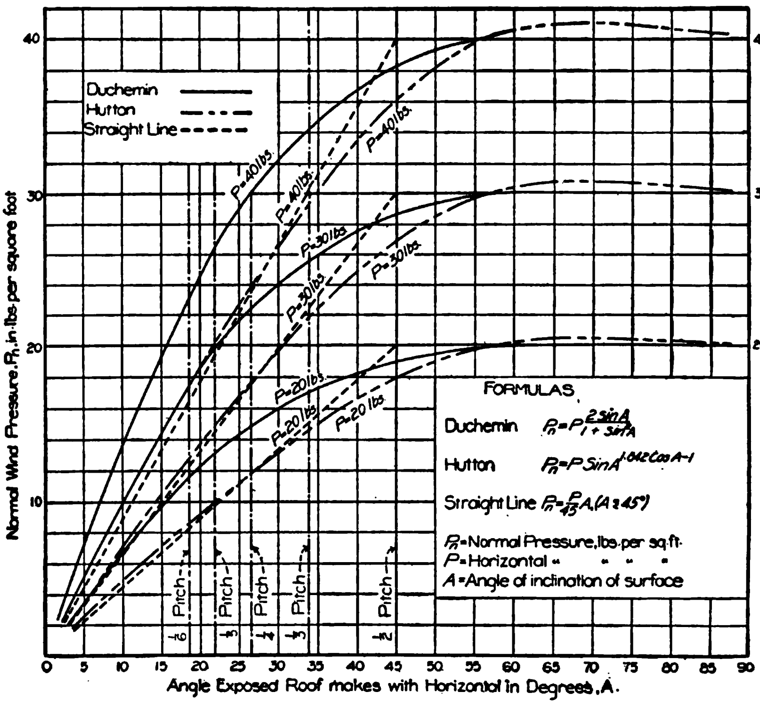

FIG. 2. NORMAL WIND LOAD ON ROOF ACCORDING TO DIFFERENT FORMULAS.

22. **Mine Buildings.**—Mine, smelter and other buildings exposed to the action of corrosive gases shall have their dead loads increased 25 per cent.

23. **Concentrated Loads.**—Concentrated loads and crane girders shall be considered in determining dead loads.

24. **Purlins.**—Purlins shall be designed to carry the actual weight of the covering, roofing and purlins, but shall always be designed for a normal load of not less than 30 lbs. per square foot.

25. **Girts.**—Girts shall be designed for a normal load of not less than 25 lbs. per square foot.

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26. **Roof Covering.**—Roof covering shall be designed for a normal load of not less than 30 lbs. per square foot.

27. **Minimum Loads.**—No roof shall, however, be designed for an equivalent load of less than 30 pounds per square foot of horizontal projection.

28. **Loads on Foundations.**—The loads on foundations shall not exceed the following in tons per square foot:

Ordinary clay and dry sand mixed with clay	2
Dry sand and dry clay	3
Hard clay and firm coarse sand	4
Firm coarse sand and gravel	5
Shale rock	8
Hard rock	20

For all soils inferior to the above, such as loam, etc., never more than one ton per square foot.

29 **Stresses in Masonry.**—The allowable stresses in masonry shall not exceed the following:

	Tons per Sq. Ft.	Lbs. per Sq. In.
Common brick, Portland cement mortar	12	168
Hard burned brick, Portland cement mortar	15	210
Rubble masonry, Portland cement mortar	10	140
First class masonry, crystalline sandstone or limestone	25	350
First class masonry, granite	30	420
Portland cement concrete, 1–3–5	20	280
Portland cement concrete, 1–2–4	30	420

30. **Pressures on Masonry.**—The pressure of column bases, beams, etc., on masonry shall not exceed the following in pounds per square inch.

Brick work with cement mortar	250
Rubble masonry with cement mortar	250
Portland cement concrete, 1–2–4	500
First class dimension sandstone or limestone	400
First class granite	500

31. **Loads on Timber Piles.**—The maximum load carried by a pile shall not exceed 40,000 lbs., or 600 lbs. per sq. in. of its average cross-section. The allowable load on piles driven with a drop hammer shall be determined by the formula $P=\frac{2W\cdot h}{s+1}$. Where $P=$ safe load on

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pile in tons; W = weight of hammer in tons; h = free fall of hammer in feet; s = average penetration for the last six blows of the hammer in inches. Where a steam hammer is used, $\frac{1}{10}$ is to be used in place of unity in the denominator of the right hand member of the formula.

Piles shall have a penetration of not less than 10 ft. in hard material, such as gravel, and not less than 15 ft. in loam or soft material.

Proportion of Parts.

32. **Allowable Stresses.**—In proportioning the different parts of the structure the maximum stresses due to the combinations of the dead and wind load; dead and snow load; or dead, minimum snow and wind load are to be provided for. Concentrated loads where they occur must be provided for.

33. **Tensile Stress.**—Allowable Unit Tensile Stresses for Structural Steel. For direct dead, snow and wind loads.

	Lbs. per Sq. In.
Shapes, main members, net section	16,000
Bars	16,000
Bottom flanges of rolled beams	16,000
Shapes, laterals, net section	20,000
Iron rods for laterals	20,000
Plate girder webs, shear on net section	10,000
Shapes liable to sudden loading as when used for crane girders	10,000
Expansion rollers per lineal inch where D = diameter of roller in inches.	$600 \times D$

Laterals shall be designed for the maximum stresses due to 5,000 pounds initial tension and the maximum stress due to wind.

34. **Compressive Stress.**—Allowable Unit Compressive Stress for Structural Steel. For direct dead, snow and wind loads

$$S = 16{,}000 - 70\frac{l}{r}$$

where S = allowable unit stress in pounds per sq. in.;
l = length of member in inches c. to c. of end connections;
r = least radius of gyration of the member in inches.

35. **Plate Girders.**—Top flanges of plate girders shall have the same gross area as the tension flanges.

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36. Shear in webs of plate girders shall not exceed 10,000 pounds per sq. in. of net section.

37. **Alternate Stress.**—Members and connections subject to alternate stresses shall be designed to take each kind of stress.

38. **Combined Stress.**—Members subject to combined direct and bending stresses shall be proportioned according to the following formula:

$$S = \frac{P}{A} + \frac{M \cdot y_1}{I \pm \frac{P \cdot l^2}{10E}}$$

where S = stress in lbs. per sq. in. in extreme fiber;
P = direct load in lbs.;
A = area of member in sq. in.;
M = bending moment in in.-lbs.;
y_1 = distance from neutral axis to extreme fiber in in.;
I = moment of inertia of member;
l = length member, or distance from point of zero moment to end of member in in.;
E = modulus of elasticity = 30,000,000.

When combined direct and flexural stress due to wind is considered, 50 per cent may be added to the above allowable tensile and compressive stresses.

39. **Stress Due to Weight of Member.**—Where the stress due to the weight of the member or due to an eccentric load exceeds the allowable stress for direct loads by more than 10 per cent, the section shall be increased until the total stress does not exceed the above allowable stress for direct loads by more than 10 per cent.

The eccentric stress caused by connecting angles by one leg when used as ties or struts shall be calculated, or only one leg will be considered effective.

40. **Rivets.**—Rivets shall be so spaced that the shearing stress shall not exceed 11,000 pounds per square inch; nor the pressure on the bearing surface (diameter × thickness of piece) of the rivet hole exceed 22,000 pounds per square inch

Rivets in lateral connections may have stresses 25 per cent in excess of the above.

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Field rivets shall be spaced for stresses two thirds those allowed for shop rivets.

Field bolts, when allowed, shall be spaced for stresses two thirds those allowed for field rivets.

Rivets and field bolts must not be used in direct tension. Where it is necessary that connections take tension turned bolts shall be used.

41. **Pins.**—Pins shall be proportioned so that the shearing stress shall not exceed 11,000 pounds per square inch; nor the pressure on the bearing surface (diameter $\times$ thickness of piece) of the pin hole exceed 22,000 pounds per square inch; nor the extreme fiber stress due to cross bending exceed 24,000 pounds per square inch when the applied forces are assumed as acting at the center of the members.

42. **Plate Girders.**—Plate girders shall be proportioned on the assumption that $\frac{1}{8}$ of the gross area of the web is available as flange area, and the shear is resisted by the web. The distance between centers of gravity of the flange areas shall be considered as the effective depth of the girder.

43. **Web Stiffeners.**—The web of plate girders shall have stiffeners at the ends and inner edges of bearing plates, and at points of concentrated loads, and also at intermediate points where the thickness of the web is less than $1/60$ of the unsupported distance between flange angles, not further apart than the depth of the full web plate with a maximum limit of 5 feet. Stiffeners shall be designed as columns for a length equal to one half the depth of the girder. Stiffener angles must have enough rivets to properly transmit the shear.

44. Compression flanges of plate girders shall have at least the same sectional area as the tension flanges, and shall not have a stress per sq. in. on the gross area greater than $16{,}000 - 150\frac{l}{b}$, where l = unsupported distance, and b = width of flange. Compression flanges of plate girders shall be stayed transversely when their length is more than thirty times their width.

45. **Rolled Beams.**—Rolled beams shall be proportioned by their moment of inertia. The depth of rolled beams in floors shall not be less than $1/20$ of the span. Where rolled beams or channels are used as roof purlins the depths shall not be less than $1/40$ of the span.

46. **Timber.**—The allowable stresses in timber purlins and other timbers shall be taken from Table II.

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TABLE II.

ALLOWABLE WORKING UNIT STRESSES, IN POUNDS, PER SQUARE INCH.

Kind of Timber.	Transverse Loading, S	End Bearing.	Columns Under 10 Diameters, C	Bearing Across Fiber.	Shear.		Modulus of Elasticity, E
					Parallel to Grain.	Longitudinal Shear in Beams.	
White Oak	1,200	1,200	1,000	450	200	110	1,150,000
Long Leaf Yellow Pine	1,300	1,300	1,000	300	180	120	1,610,000
White Pine and Spruce	1,000	1,000	800	200	100	70	1,130,000
Western Hemlock	1,000	1,000	800	200	160	100	1,480,000
Douglass Fir	1,200	1,200	1,000	350	180	110	1,510,000

Columns may be used with a length not exceeding 45 times the least dimension. The unit stress for lengths of more than 10 times the least dimension shall be reduced by the following formula:

$$P = C - \frac{C}{100}\frac{l}{d}$$

where C = unit stress, as given above for short columns;
P = allowable unit stress in lbs. per sq. in.;
l = length of column in in.;
d = least side of column in in.

COVERING.

47. **Corrugated Steel.**—Corrugated steel shall generally have 2½ inch corrugations when used for roof and sides of buildings, and 1¼ inch corrugations when used for lining buildings. The minimum gage of corrugated steel shall be No. 22 for roofs, No. 24 for sides, and No. 26 for lining.

The gage of corrugated steel in U. S. standard gage and weight per square foot shall be shown on the general plan.

48. **Spacing Purlins and Girts.**—The span, or center to center distance of purlins, shall not exceed the distance given in Fig. 3 for a safe load of 30 lbs. per sq. ft. Corrugated steel sheets shall preferably span two purlin spaces. Girts shall be spaced for a safe load of 25 lbs. per sq. ft. in Fig. 3.

49. **End and Side Laps.**—Corrugated steel shall be laid with two corrugations side lap and six inches end lap when used for roofing, and one corrugation side lap and four inches end lap when used for siding.

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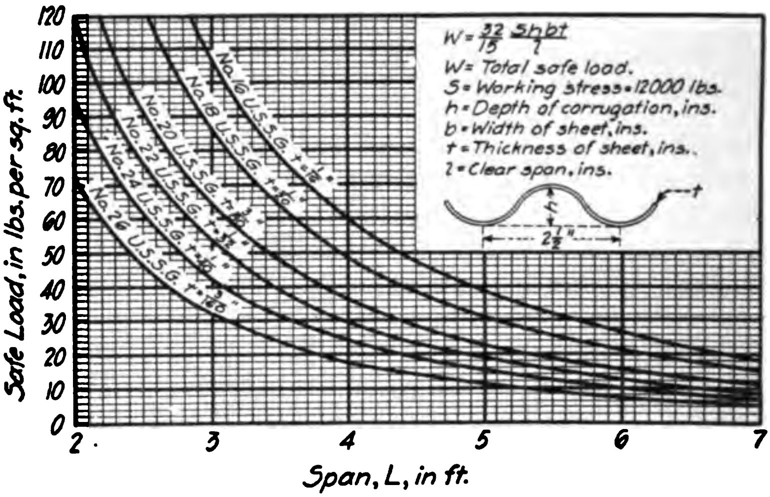

FIG. 3. SAFE UNIFORM LOAD IN POUNDS FOR CORRUGATED STEEL FOR DIFFERENT SPANS IN FEET.

50. **Fastening.**—Corrugated steel shall be fastened to the purlins and girts by means of galvanized iron straps ¾ inch wide by No. 18 gage, spaced 8 to 12 inches apart; by clinch nails spaced 8 to 12 inches apart; or by nailing directly to spiking strips with 8d barbed nails, spaced 8 inches apart. Spiking strips shall preferably be used with anti-condensation lining. Bolts, nails and rivets shall always pass through the top of corrugations. Side laps shall be riveted with copper or galvanized iron rivets 8 to 12 inches apart on the roof and 1½ to 2 feet apart on the sides.

51. **Corrugated Steel Lining.**—Corrugated steel lining on the sides shall be laid with one corrugation side lap and four inches end lap. Girts for corrugated steel lining shall be spaced for a safe load of 25 pounds per square foot as given in Fig. 3.

52. **Anti-condensation Lining.**—Anti-condensation roof lining shall be used to prevent dripping in engine houses and similar buildings, and shall be constructed as follows: Galvanized wire poultry netting is fastened to one eave purlin and is passed over the ridge, stretched tight and fastened to the other eave purlin. The edges of the wire are woven together and the netting is fastened to the spiking strips, where used, by means of small staples. On the netting are laid two layers of asbestos paper $\frac{1}{16}$ in. thick and two layers of tar paper. The corru-

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gated steel is then fastened to the purlins in the usual way; $\frac{3}{16}$ in. stove bolts with $1'' \times \frac{1}{8}'' \times 4''$ plate washers on the lower side are used for fastening the side laps together and for supporting the lining; or the purlins may be spaced one half the usual distance where anti-condensation lining is used and the stove bolts omitted.

53. **Flashing.**—Valleys or corners around stacks shall have flashing extending at least 12 inches above where water will stand, and shall be riveted or soldered, if necessary, to prevent leakage.

Flashing shall be provided above doors and windows.

54. **Ridge Roll.**—All ridges shall have a ridge roll securely fastened to the corrugated steel.

55. **Corner Finish.**—All corners shall be covered with standard corner finish securely fastened to the corrugated steel.

56. **Cornice.**—At the gable ends the corrugated steel on the roof shall be securely fastened to a finish angle or channel connected to the end of the purlins, or, where molded cornices are used, to a piece of timber fastened to the ends of the purlins.

57. **Gutters.**—Gutters and conductors shall be furnished at least equal to the requirements of the following table:

Span of Roof.	Gutter.	Conductor
Up to 50 ft.	6 in.	4 in. every 40 ft.
50 ft. to 70 ft.	7 in.	5 in. every 40 ft.
70 ft. to 100 ft.	8 in.	5 in. every 40 ft.

Gutters shall have a slope of at least 1 in. in 15 ft. Gutters and conductors shall be made of galvanized steel not lighter than No. 24.

58. **Ventilators.**—Ventilators shall be provided and located so as to properly ventilate the building. They shall have a net opening for each 100 square feet of floor space as follows: not less than one fourth square foot for clean machine shops and similar buildings; not less than one square foot for dirty machine shops; not less than four square feet for mills; and not less than six square feet for forge shops, foundries and smelters.

59. **Shutters and Louvres.**—Openings in ventilators shall be provided with shutters, sash, or louvres, or may be left open as specified.

Shutters must be provided with a satisfactory device for opening and closing.

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Louvres must be designed to prevent the blowing in of rain and snow, and must be made stiff so that no appreciable sagging will occur. They shall be made of not less than No. 20 gage galvanized steel for flat louvres, and No. 24 gage galvanized steel for corrugated louvres.

60. **Circular Ventilators.**—Circular ventilators, when used, must be designed so as to prevent down drafts. Net opening only shall be used in calculations.

61. **Windows.**—Windows shall be provided in the exterior walls equal to not less than 10 per cent of the entire exterior surface in mill buildings, and of not less than 25 per cent in machine shops, factories, washeries, concentrators, breakers and similar buildings.

Window glass up to 12 in. $\times$ 14 in. may be single strength, over 12 in. $\times$ 14 in. the glass shall be double strength. Window glass shall be A grade except in smelters, foundries, forge shops and similar structures, where it may be B grade. The sash and frames shall be constructed of white pine. Where buildings are exposed to fire hazard the windows shall have wire glass set in metal sash and frames.

62. **Skylights.**—At least half of the lighting shall preferably be by means of skylights, or sash in the sides of ventilators.

Skylights shall be glazed with wire glass, or wire netting shall be stretched beneath the skylights to prevent the broken glass from falling into the building. Where there is danger of the skylight glass being broken by objects falling on it, a wire netting guard shall be provided on the outside.

Skylight glass shall be carefully set, special care being used to prevent leakage. Leakage and condensation on the inner surface of the glass shall be carried to the down-spouts, or outside the building by condensation gutters.

63. Windows in sides of buildings shall be made with counterbalanced sash, and in ventilators shall be made with sliding or swing sash. All swinging windows shall be provided with a satisfactory operating device.

64. **Doors.**—Doors are to be furnished as specified and are to be provided with hinges, tracks, locks and bolts. Single doors up to 4 feet and double doors up to 8 feet shall preferably be swung on hinges; large doors, double and single, shall be arranged to slide on overhead tracks, or may be counterbalanced to lift up between vertical guides.

Steel doors shall be firmly braced and shall be covered with No. 24 corrugated steel with 1¼ in. corrugations.

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The frames of sandwich doors shall be made of two layers of $\frac{7}{8}$ in. matched white pine, placed diagonally, and firmly nailed with clinch nails. The frame shall be covered on each side with a layer of No. 26 corrugated steel with $1\frac{1}{4}$ in corrugations. Locks and all other necessary hardware shall be furnished for all windows and doors.

65. **TAR AND GRAVEL ROOF.**—Tar and gravel roofs are called three-, four-, five-ply, etc., depending upon the number of layers of roofing felt. Tar and gravel roofs may be laid upon timber sheathing or upon concrete slabs.

66. **Specifications for Five-Ply Tar and Gravel Roof on Board Sheathing.**—The materials used in making the roof are 1 (one) thickness of sheathing paper or unsaturated felt, 5 (five) thicknesses of saturated felt weighing not less than 15 (fifteen) pounds per square of one hundred (100) square feet, single thickness, and not less than one hundred and twenty (120) pounds of pitch, and not less than four hundred (400) pounds of gravel or three hundred (300) pounds of slag from $\frac{1}{4}$ to $\frac{5}{8}$ in. in size, free from dirt, per square of one hundred (100) square feet of completed roof.

67. The material shall be applied as follows: First, lay the sheathing or unsaturated felt, lapping each sheet one inch over the preceding one. Second, lay two (2) thicknesses of tarred felt, lapping each sheet seventeen (17) inches over the preceding one, nailing as often as may be necessary to hold the sheets in place until the remaining felt is applied. Third, coat the entire surface of this two-ply layer with hot pitch, mopped on uniformly. Fourth, apply three (3) thicknesses of felt, lapping each sheet twenty-two (22) inches over the preceding one, mopping with hot pitch the full width of the 22 inches between the plies, so that in no case shall felt touch felt. Such nailing as is necessary shall be done so that all nails will be covered by not less than two plies of felt; fifth, spread over the entire surface of the roof a uniform coating of pitch, into which, while hot, imbed the gravel or slag. The gravel or slag in all cases must be dry.

68. **Specifications for Five-Ply Tar and Gravel Roof on Concrete Sheathing.**—The materials used shall be the same as for tar and gravel roof on timber sheathing, except that the one thickness of sheathing paper or unsaturated felt may be omitted.

69. The materials shall be applied as follows: First, coat the concrete with hot pitch, mopped on uniformly. Second, lay two (2) thick-

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nesses of tarred felt, lapping each sheet seventeen (17) inches over the preceding one, and mop with hot pitch the full width of the 17 inch lap, so that in no case shall felt touch felt. Third, coat the entire surface with hot pitch, mopped on uniformly. Fourth, lay three (3) thicknesses of felt, lapping each sheet twenty-two (22) inches over the preceding one, mopping with hot pitch the full width of the 22 inch lap between the plies, so that in no case shall felt touch felt. Fifth, spread the entire surface of the roof with a uniform coat of pitch, into which, while hot, imbed gravel or slag.

70. **SPECIFICATIONS FOR CEMENT FLOOR ON A CONCRETE BASE. Materials.**—The cement used shall be first-class Portland cement, and shall pass the standards of the American Society for Testing Materials. The sand for the top finish shall be clean and sharp and shall be retained on a No. 30 sieve and shall have passed the No. 20 sieve. Broken stone for the top finish shall pass a ½ in. screen and shall be retained on the No. 20 screen. Dust shall be excluded. The sand for the base shall be clean and sharp. The aggregate for the base shall be of broken stone or gravel and shall pass a 2 in. ring.

71. **Base.**—On a thoroughly tamped and compacted subgrade the concrete for the base shall be laid and thoroughly tamped. The base shall not be less than 2½ in. thick. Concrete for the base shall be thoroughly mixed with sufficient water so that some tamping is required to bring the moisture to the surface. If old concrete is used for the base the surface shall be roughened and thoroughly cleaned so that the new mortar will adhere. The roughened surface of old concrete shall then be thoroughly wet so that the base will not draw water from the finish when the latter is applied. Before scrubbing the base with grout the excess water shall be removed.

72. **Finish.**—With old concrete the surface of the base shall first be scrubbed with a thin grout of pure cement, rubbed in with a broom. On top of this, before the thin coat is set, a coat of finish mixed in the proportions of one part Portland cement, one part stone broken to pass a ½ in. ring, and one part sand shall be troweled on using as much pressure as possible, so that it will take a firm bond. After the finish has been applied to the desired thickness it should be screeded and floated to a true surface. Between the time of initial and final set it shall be finished by skilled workmen with steel trowels and shall be worked to final surface. Under no condition shall a dryer be used, nor shall water be added to make the material work easily.

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73. **SPECIFICATIONS FOR WOOD FLOOR ON A TAR CONCRETE BASE. Floor Sleepers.**—Sleepers for carrying the timber floor shall be 3 in. $\times$ 3 in. placed 18 in. c. to c. After the subgrade has been thoroughly tamped and rolled to an elevation of $4\frac{1}{2}$ in. below the tops of the sleepers, the sleepers shall be placed in position and supported on stakes driven in the subgrade. Before depositing the tar concrete the sleepers must be brought to a true level.

74. **Tar Concrete Base.**—The tar concrete base shall be not less than $4\frac{1}{2}$ in. thick and shall be laid as follows: First, a layer three (3) inches thick of coarse, screened gravel thoroughly mixed with tar, and tamped to a hard level surface. Second, on this bed spread a top dressing $1\frac{1}{2}$ inches thick of sand heated and thoroughly mixed with coal tar pitch, in the proportions of one (1) part pitch to three (3) parts tar. The gravel, sand and tar shall be heated to from 200 to 300 degrees F., and shall be thoroughly mixed and carefully tamped into place.

75. **Plank Sub-Floor.**—The floor plank shall be of sound hemlock or pine not less than 2 inches thick, planed on one side and one edge to an even thickness and width. The floor plank is to be toe-nailed with 4 in. wire nails.

76. **Finished Flooring.**—The finished flooring is to be of maple of clear stock, $\frac{7}{8}$ in. finished thickness, thoroughly air and kiln dried and not over 4 inches wide. The floor is to be planed to an even thickness, the edges jointed, and the underside channeled or ploughed. The finished floor is to be laid at right angles to the sub-floor, and each board neatly fitted at the ends, breaking joints at random. The floor is to be final nailed with 10 d. or 3 in. wire nails, nailed in diagonal rows 16 inches apart across the boards, with two (2) nails in each row in every board. The floor to be finished off perfectly smooth on completion.

77. The finished flooring is not to be taken into the building or laid until the tar concrete base and sub-plank floor are thoroughly dried.

Details of Construction.

78. **Details.**—All connections and details shall be of sufficient strength to develop the full strength of the member.

79. **Pitch of Rivets.**—The pitch of rivets shall not exceed 6 inches, or sixteen times the thickness of the thinnest outside plate in the line

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of stress, nor forty times the thickness of the thinnest outside plate at right angles to the line of stress. The pitch shall never be less than three diameters of rivet. At the ends of compression members the pitch shall not exceed four diameters of the rivet for a length equal to twice the width of the member.

80. **Edge Distance.**—The minimum distance from the center of any rivet hole to a sheared edge shall be $1\frac{1}{2}$ in. for $\frac{7}{8}$ in. rivets, $1\frac{1}{4}$ in. for $\frac{3}{4}$ in. rivets, $1\frac{1}{8}$ in. for $\frac{5}{8}$ in. rivets, and 1 in. for $\frac{1}{2}$ in. rivets, and to a rolled edge $1\frac{1}{4}$, $1\frac{1}{8}$, 1 and $\frac{7}{8}$ in., respectively. The maximum distance from the edge shall be eight (8) times the thickness of the plate.

81. **Maximum Diameter.**—The diameter of the rivets in angles carrying calculated stresses shall not exceed $\frac{1}{4}$ of the width of the leg in which they are driven, except that $\frac{5}{8}$ in. rivets may be used in 2 in. angles.

82. **Diameter of Punch and Die.**—The diameter of the punch and die shall be as specified in § 147.

83. **Net Sections.**—The effective diameter of a driven rivet will be assumed the same as its diameter before driving. In deducting the rivet holes to obtain net sections in tension members, the diameter of the rivet holes will be assumed as $\frac{1}{8}$ inch larger than the undriven rivet.

84. **Minimum Sections.**—No metal of less thickness than $\frac{1}{4}$ in. shall be used except for fillers; and no angles less than $2'' \times 2'' \times \frac{1}{4}''$ The minimum thickness of metal in head frames, rock houses and coal tipples, coal washers and coal breakers shall be $\frac{5}{16}$ inch, except for fillers. No upset rod shall be less than $\frac{5}{8}$ in. in diameter. Sag rods may be as small as $\frac{3}{8}$ in. diameter.

85. **Connections.**—All connections shall be of sufficient strength to develop the full strength of the member. No connections except for lacing bars shall have less than two rivets. All field connections except lacing bars shall have not less than three rivets.

86. **Flange Plates.**—The flange plates of all girders shall not extend beyond the outer line of rivets connecting them to the angles more than six inches nor more than eight times the thickness of the thinnest plate.

87. **Web Stiffeners.**—Web stiffeners shall be in pairs, and shall have a close fit against flange angles. The stiffeners at the ends of plate girders shall have filler plates. Intermediate stiffeners may have fillers or be crimped over the flange angles. The rivet pitch in stiffeners shall not be greater than 5 inches.

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88. **Web Splices.**—Web plates shall be spliced at all points by a plate on each side of the web, capable of transmitting the shearing and bending stresses through the splice rivets.

89. **Net Sections.**—Net sections must be used in calculating tension members and in deducting the rivet holes they shall be taken $\frac{1}{8}$ in. larger than the nominal size of rivet.

90. Pin connected riveted tension members shall have a net section through the pin hole 25 per cent in access of the required net section of the member. The net section back of the pin hole in line of the center of the pin shall be at least 0.75 of the net section through the pin hole.

91. **Upset Rods.**—All rods with screw ends, except sag rods, must be upset at the ends so that the diameter at the base of the threads shall be $\frac{1}{16}$ inch larger than any part of the body of the bar.

92. **Upper Chords.**—Upper chords of trusses shall have symmetrical cross-sections, and shall preferably consist of two angles back to back.

93. **Compression Members.**—All other compression members for roof trusses, except sub-struts, shall be composed of sections symmetrically placed. Sub-struts may consist of a single section.

94. **Columns.**—Side posts which take flexure shall preferably be composed of 4 angles laced, or 4 angles and a plate. Where side posts do not take flexure and carry heavy loads they shall preferably be composed of two channels laced, or of two channels with a center diaphragm.

95. Posts in end framing shall preferably be composed of I-beams or 4 angles laced. Corner columns shall preferably be composed of one angle.

96. **Crane Posts.**—The cross-bending stress due to eccentric loading in columns carrying cranes shall be calculated. Crane girders carrying heavy cranes shall be carried on independent columns.

97. **Batten Plates.**—Laced compression members shall be stayed at the ends by batten plates, placed as near the end of the member as practicable and having a length not less than the greatest width of the member. The thickness of batten plates shall not be less than $1/40$ of the distance between rivet lines at right angles to axis of member.

98. **Lacing.**—Single lacing bars shall have a thickness of not less than $1/40$, and double bars connected by a rivet at the intersection of not less than $1/60$ of the distance between the rivets connecting them to

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the member; they shall make an angle not less than 45 degrees with the axis of the member; their width shall be in accordance with the following standards, generally:

Size of Member.	Width of Lacing Bars.
For 15 inch channels, or built sections with $3\frac{1}{2}$ and 4 inch angles.	$2\frac{1}{2}$ inches ($\frac{7}{8}$ inch rivets).
For 12, 10, 9 inch channels, or built sections with 3 inch angles.	$2\frac{1}{4}$ inches ($\frac{3}{4}$ inch rivets).
For 8 and 7 inch channels, or built sections with $2\frac{1}{2}$ inch angles.	2 inches ($\frac{5}{8}$ inch rivets).
For 6 and 5 inch channels, or built sections with 2 inch angles.	$1\frac{3}{4}$ inches ($\frac{1}{2}$ inch rivets).

Where laced members are subjected to bending, the size of lacing bars or angles shall be calculated, or a solid web plate shall be used.

99. **Pin Plates.**—All pin holes shall be reinforced by additional material when necessary, so as not to exceed the allowable pressure on the pins. These reinforcing plates must contain enough rivets to transfer the proportion of pressure which comes upon them, and at least one plate on each side shall extend not less than 6 inches beyond the edge of the batten plate.

100. **Maximum Length of Compression Members.**—No compression member shall have a length exceeding 125 times its least radius of gyration for main members, nor 150 times its least radius of gyration for laterals and sub-members. The length of a main tension member in which the stress is reversed by the wind shall not exceed 150 times its least radius of gyration.

101. **Maximum Length of Tension Members.**—The length of riveted tension members in horizontal or inclined position shall not exceed 200 times their radius of gyration except for wind bracing, which members may have a length equal to 250 times the least radius of gyration. The horizontal projection of the unsupported portion of the member is to be considered the effective length.

102. **Splices.**—In compression members joints with abutting faces planed shall be placed as near the panel points as possible, and must be spliced on all sides with at least two rows of rivets on each side of the joint. Joints with abutting faces not planed must be fully spliced.

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103. **Splices.**—Joints in tension members shall be fully spliced.

104. **Tension Members.**—Tension members shall preferably be composed of angles or shapes capable of taking compression as well as tension. Flats riveted at the ends shall not be used.

105. Main tension members shall preferably be made of 2 angles, 2 angles and a plate, or 2 channels laced. Secondary tension members may be made of a single shape.

106. **Eye-Bars.**—Heads of eye-bars shall be so proportioned as to develop the full strength of the bar. The heads shall be forged and not welded.

107. **Pins.**—Pins must be turned true to size and straight, and must be driven to place by means of pilot nuts.

The diameter of pin shall not be less than ¾ of the depth of the widest bar attached to it.

The several members attached to a pin shall be packed so as to produce the least bending moment on the pin, and all vacant spaces must be filled with steel or cast iron fillers.

108. **Bars or Rods.**—Long laterals may be made of bars with clevis or sleeve nut adjustment. Bent loops shall not be used.

109. **Spacing Trusses.**—Trusses shall preferably be spaced so as to allow the use of single pieces of rolled sections for purlins. Trussed purlins shall be avoided if possible.

110. **Purlins and Girts.**—Purlins and girts shall preferably be composed of single sections—channels, angles or Z-bars, placed with web at right angles to the trusses and posts and legs turned down.

111. **Fastening.**—Purlins and girts shall be attached to the top chord of trusses and to columns by means of angle clips with two rivets in each leg.

112. **Spacing.**—Purlins for corrugated steel without sheathing shall be spaced at distances apart not to exceed the span as given for a safe load of 30 pounds, and girts for a safe load of 25 pounds as given in Fig. 3.

113. **Timber Purlins.**—Timber purlins and girts shall be attached and spaced the same as steel purlins.

114. **Base Plates.**—Base plates shall never be less than ½ inch in thickness, and shall be of sufficient thickness and size so that the pressure on the masonry shall not exceed the allowable pressures in § 30.

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115. **Anchors.**—Columns shall be anchored to the foundations by means of two anchor bolts not less than 1 in in diameter upset, placed as wide apart as practicable in the plane of the wind. The anchorage shall be calculated to resist one and one half times the bending moment at the base of the columns.

116. **Lateral Bracing.**—Lateral bracing shall be provided in the plane of the top and bottom chords, sides and ends; knee braces in the transverse bents; and sway bracing wherever necessary. Lateral bracing shall be designed for an initial stress of 5,000 pounds in each member, and provision must be made for putting this initial stress into the members in erecting.

117. **Temperature.**—Variations in temperature to the extent of 150 degrees F. shall be provided for.

MATERIAL AND WORKMANSHIP.

Material.

118. **Process of Manufacture.**—Steel shall be made by the open-hearth process.

119. **Schedule of Requirements.**

Chemical and Physical Properties.	Structural Steel.	Rivet Steel.	Steel Castings.
Phosphorus Max.. Basic ...	0.04 per cent.	0.04 per cent.	0.05 per cent.
Phosphorus Max.. Acid.....	0.08 " "	0.04 " "	0.08 " "
Sulphur maximum...........	0.05 " "	0 04 " "	0.05 " "
Ultimate tensile strength.. Pounds per square inch......	Desired 60,000	Desired 50,000	Not less than 65,000
Elongation: min. % in 8"	1,500,000* / Ult. tensile strength	1,500,000 / Ult. tensile strength.	
Elongation: min. % in 2"...	22		18
Character of fracture.........	Silky	Silky	Silky or fine granular
Cold bends without fracture.	180° flat †	180° flat ‡	90°, $d = 3t$

The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

120. **Allowable Variations.**—If the ultimate strength varies more than 4,000 lbs. from that desired, a retest shall be made on the same

* See paragraph 128.

† See paragraphs 129, 130 and 131.

‡ See paragraph 132.

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gage, which, to be acceptable, shall be within 5,000 lbs. of the desired ultimate.

121. **Chemical Analyses.**—Chemical determinations of the percentages of carbon, phosphorus, sulphur and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for by the purchaser, in which case an excess of 25 per cent above the required limits will be allowed.

122. **Form of Specimens.** PLATES, SHAPES AND BARS.—Specimens for tensile and bending tests for plates, shapes and bars shall be made by cutting coupons from the finished product, which shall have both faces rolled and both edges milled to the form shown by Fig. 1; or with both edges parallel; or they may be turned to a diameter of ¾ inch for a length of at least 9 inches, with enlarged ends.

123. RIVETS.—Rivet rods shall be tested as rolled.

124. PINS AND ROLLERS.—Specimens shall be cut from the finished rolled or forged bar, in such manner that the center of the specimen shall be 1 inch from the surface of the bar. The specimen for tensile test shall be turned to the form shown by Fig. 2. The specimen for bending test shall be 1 inch by ½ inch in section.

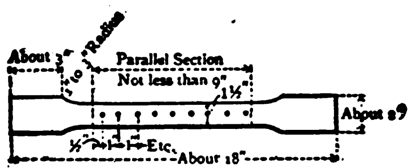

FIG. 1

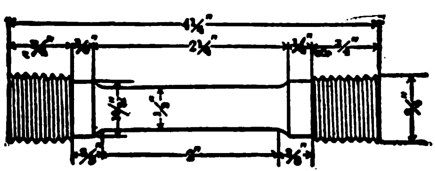
FIG. 2.

125. STEEL CASTINGS.—The number of tests will depend on the character and importance of the castings. Specimens shall be cut cold from coupons molded and cast on some portion of one or more castings from each melt or from the sink heads, if the heads are of sufficient

size. The coupon or sink head, so used, shall be annealed with the casting before it is cut off. Test specimens shall be of the form prescribed for pins and rollers.

126. **Annealed Specimens.**—Material which is to be used without annealing or further treatment shall be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimens for tensile tests representing such material shall be cut from properly annealed or similarly treated short lengths of the full section of the bar.

127. **Number of Tests.**—At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing $\frac{3}{8}$ in. and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

128. **Modifications in Elongation.**—For material less than $\frac{5}{16}$ in. and more than $\frac{3}{4}$ in. in thickness the following modifications will be allowed in the requirements for elongation:

(*a*) For each $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in., a deduction of $2\frac{1}{2}$ per cent will be allowed from the specified elongation.

(*b*) For each $\frac{1}{8}$ in. in thickness above $\frac{3}{4}$ in., a deduction of 1 per cent will be allowed from the specified elongation.

(*c*) For pins and rollers over 3 in. in diameter the elongation in 8 in. may be 5 per cent less than that specified in paragraph 119.

129. **Bending Tests.**—Bending tests may be made by pressure or by blows. Plates, shapes and bars less than 1 in. thick shall bend as called for in paragraph 119.

130. **Thick Material.**—Full-sized material for eye-bars and other steel 1 in. thick and over, tested as rolled, shall bend cold 180 degrees around a pin the diameter of which is equal to twice the thickness of the bar, without fracture on the outside of bend.

131. **Bending Angles.**—Angles $\frac{3}{4}$ in. and less in thickness shall open flat and angles $\frac{1}{2}$ in. and less in thickness shall bend shut, cold, under blows of a hammer, without sign of fracture. This test will be made only when required by the inspector.

132. **Nicked Bends.**—Rivet steel, when nicked and bent around a bar of the same diameter as the rivet rod, shall give a gradual break and a fine, silky, uniform fracture.

133. **Finish.**—Finished material shall be free from injurious seams, flaws, cracks, defective edges, or other defects, and have a smooth,

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uniform, workmanlike finish. Plates 36 in. in width and under shall have rolled edges.

134. **Stamping.**—Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it. Steel for pins and rollers shall be stamped on the end. Rivet and lattice steel and other small parts may be bundled with the above marks on an attached metal tag.

135. **Defective Material.**—Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious defects, will be rejected at the shop and shall be replaced by the manufacturer at his own cost.

136. **Allowable Variation in Weight.**—A variation in cross-section or weight of each piece of steel of more than 2½ per cent from that specified will be sufficient cause for rejection, except in case of sheared plates, which will be covered by the following permissible variations, which are to apply to single plates.

137. **When Ordered to Weight.**—Plates 12½ pounds per square foot or heavier:

(*a*) Up to 100 in. wide, 2½ per cent above or below the prescribed weight.

PLATES ¼ INCH AND OVER IN THICKNESS.

Thickness Ordered.	Nominal Weight.	Width of Plate.			
		Up to 75 Inch.	75″ and up to 100″.	100″ and up to 115.″	Over 115.″
1-4 inch.	10.20 lbs.	10 per cent.	14 per cent.	18 per cent.	
5-16 "	12.75 "	8 " "	12 " "	16 " "	
3-8 "	15.30 "	7 " "	10 " "	13 " "	17 per cent.
7-16 "	17.85 "	6 " "	8 " "	10 " "	13 " "
1-2 "	20.40 "	5 " "	7 " "	9 " "	12 " "
9-16 "	22.95 "	4½ " "	6½ " "	8½ " "	11 " "
5-8 "	25.50 "	4 " "	6 " "	8 " "	10 " "
Over 5-8 "		3½ " "	5 " "	6½ " "	9 " "

PLATES UNDER ¼ INCH IN THICKNESS.

Thickness Ordered.	Nominal Weights Lbs. per Square Ft.	Width of Plate.		
		Up to 50.″	50″ and up to 70″	Over 70.″
1-8″ up to 5-32″	5.10 to 6.37	10 per cent.	15 per cent.	20 per cent.
5-32 " " 3-16	6.37 " 7.65	8½ " "	12½ " "	17 " "
3-16 " " 1-4	7.65 " 10.20	7 " "	10 " "	15 " "

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(*b*) One hundred in. wide and over, 5 per cent above or below

138. Plates under $12\frac{1}{2}$ pounds per square foot:

(*a*) Up to 75 in. wide, $2\frac{1}{2}$ per cent above or below.

(*b*) Seventy-five in. and up to 100 in. wide, 5 per cent above or 3 per cent below.

(*c*) One hundred in. wide and over, 10 per cent above or 3 per cent below.

139. **When Ordered to Gage.**—Plates will be accepted if they measure not more than .01 inch below the ordered thickness.

140. An excess over the nominal weight, corresponding to the dimensions on the order, will be allowed for each plate, if not more than that shown in the preceding tables, one cubic inch of rolled steel being assumed to weigh 0.2833 pounds.

Special Metals.

141. **Cast-Iron.**—Except where chilled iron is specified, castings shall be made of tough gray iron, with sulphur not over 0.10 per cent. They shall be true to pattern, out of wind and free from flaws and excessive shrinkage. If tests are demanded they shall be made on the "Arbitration Bar" of the American Society for Testing Materials, which is a round bar, $1\frac{1}{4}$ in. in diameter and 15 in. long. The transverse test shall be on a supported length of 12 in. with load at middle. The minimum breaking load so applied shall be 2,900 lbs., with a deflection of at least $\frac{1}{10}$ inch before rupture.

142. **Wrought-Iron Bars.**—Wrought-iron shall be double-rolled, tough, fibrous and uniform in character. It shall be thoroughly welded in rolling and be free from surface defects When tested in specimens of the form of Fig. 1, or in full-sized pieces of the same length, it shall show an ultimate strength of at least 50,000 lbs. per sq. in., an elongation of at least 18 per cent in 8 in., with fracture wholly fibrous. Specimens shall bend cold, with the fiber through 135°, without sign of fracture, around a pin the diameter of which is not over twice the thickness of the piece tested. When nicked and bent the fracture shall show at least 90 per cent fibrous.

Workmanship.

143. **General.**—All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works.

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144 **Straightening Material.**—Material shall be thoroughly straightened in the shop, by methods that will not injure it, before being laid off or worked in any way.

145. **Finish.**—Shearing shall be neatly and accurately done and all portions of the work exposed to view neatly finished.

146. **Rivets.**—The size of rivets, called for on the plans, shall be understood to mean the actual size of the cold rivet before heating.

147. **Rivet Holes.**—When general reaming is not required, the diameter of the punch for material not over $\frac{3}{4}$ in. thick shall be not more than $\frac{1}{16}$ in., nor that of the die more than $\frac{1}{8}$ in. larger than the diameter of the rivet. The diameter of the die shall not exceed that of the punch by more than $\frac{1}{4}$ the thickness of the metal punched.

148. **Planing and Reaming.**—In medium steel over $\frac{3}{4}$ of an in. thick, all sheared edges shall be planed and all holes shall be drilled or reamed to a diameter of $\frac{1}{8}$ of an in. larger than the punched holes, so as to remove all the sheared surface of the metal. Steel which does not satisfy the drifting test must nave holes drilled.

149. **Punching.**—Punching shall be accurately done. Slight inaccuracy in the matching of holes may be corrected with reamers. Drifting to enlarge unfair holes will not be allowed. Poor matching of holes will be cause for rejection by the inspector.

150. **Assembling.**—Riveted members shall have all parts well pinned up and firmly drawn together with bolts before riveting is commenced. Contact surfaces to be painted (see § 182).

151. **Lacing Bars.**—Lacing bars shall have neatly rounded ends, unless otherwise called for.

152. **Web Stiffeners.**—Stiffeners shall fit neatly between flanges of girders. Where tight fits are called for the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

153. **Splice Plates and Fillers.**—Web splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$ in. of flange angles.

154. **Web Plates.**—Web plates of girders, which have no cover plates, shall be flush with the backs of angles or be not more than $\frac{1}{4}$ in. scant, unless otherwise called for. When web plates are spliced, not more than $\frac{1}{4}$ in. clearance between ends of plates will be allowed.

155. **Connection Angles.**—Connection angles for girders shall be flush with each other and correct as to position and length of girder.

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In case milling is required after riveting, the removal of more than $\frac{1}{16}$ in. from their thickness will be cause for rejection.

156. **Riveting.**—Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

157. Rivets shall look neat and finished, with heads of approved shape, full and of equal size. They shall be central on shank and grip the assembled pieces firmly. Recupping and calking will not be allowed. Loose, burned or otherwise defective rivets shall be cut out and replaced. In cutting out rivets great care shall be taken not to injure the adjacent metal. If necessary they shall be drilled out.

158. **Turned Bolts.**—Wherever bolts are used in place of rivets which transmit shear, the holes shall be reamed parallel and the bolts turned to a driving fit. A washer not less than $\frac{1}{4}$ in. thick shall be used under nut.

159. **Members to be Straight.**—The several pieces forming one built member shall be straight and fit closely together, and finished members shall be free from twists, bends or open joints.

160. **Finish of Joints.**—Abutting joints shall be cut or dressed true and straight and fitted close together, especially where open to view. In compression joints depending on contact bearing the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

161. **Field Connections.**—All holes for field rivets in splices in tension members carrying live loads shall be accurately drilled to an iron templet or reamed while the connecting parts are temporarily put together.

162. **Eye-Bars.**—Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to break in the body with a silky fracture, when tested to rupture. The thickness of head and neck shall not vary more than $\frac{1}{16}$ in. from the thickness of the bar.

163. **Boring Eye-Bars.**—Before boring, each eye-bar shall be properly annealed and carefully straightened. Pin holes shall be in the center line of bars and in the center of heads. Bars of the same length

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shall be bored so accurately that, when placed together, pins $\frac{1}{32}$ in. smaller in diameter than the pin holes can be passed through the holes at both ends of the bars at the same time.

164. **Pin Holes.**—Pin holes shall be bored true to gage, smooth and straight; at right angles to the axis of the member and parallel to each other, unless otherwise called for. Wherever possible, the boring shall be done after the member is riveted up.

165. The distance center to center of pin holes shall be correct within $\frac{1}{32}$ in., and the diameter of the hole not more than $\frac{1}{50}$ in. larger than that of the pin, for pins up to 5 in. diameter, and $\frac{1}{32}$ in. for larger pins.

166. **Pins and Rollers.**—Pins and rollers shall be accurately turned to gage and shall be straight and smooth and entirely free from flaws.

167. **Pilot Nuts.**—At least one pilot and one driving nut shall be furnished for each size of pin for each structure, and field rivets 10 per cent in excess of the number of each size actually required.

168 **Screw Threads.**—Screw threads shall make tight fits in the nuts and shall be U. S standard, except above the diameter of $1\frac{3}{8}$ in., when they shall be made with six threads per in.

169. **Annealing.**—Steel, except in minor details, which has been partially heated shall be properly annealed.

170. **Steel Castings.**—All steel castings shall be annealed.

171. **Welds.**—Welds in steel will not be allowed.

172. **Bed Plates.**—Expansion bed plates shall be planed true and smooth. Cast wall plates shall be planed top and bottom. The cut of the planing tool shall correspond with the direction of expansion.

173. **Shipping Details.**—Pins, nuts, bolts, rivets, and other small details shall be boxed or crated.

174. **Weight.**—The weight of every piece and box shall be marked on it in plain figures.

175. **Finished Weight.**—Payment for pound price contracts shall be by scale weight. No allowance over 2 per cent of the actual total weight of the structure as computed from the shop plans will be allowed for excess weight.

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Additional Specifications When General Reaming and Planing are Required.

176. **Planing Edges.**—Sheared edges and ends shall be planed off at least $\frac{1}{4}$ in.

177. **Reaming.**—Punched holes shall be made with a punch $\frac{3}{16}$ in. smaller in diameter than the nominal size of the rivets and shall be reamed to a finished diameter of not more than $\frac{1}{16}$ in. larger than the rivet.

178. **Reaming after Assembling.**—Wherever practicable, reaming shall be done after the pieces forming one built member have been assembled and firmly bolted together. If necessary to take the pieces apart for shipping and handling, the respective pieces reamed together shall be so marked that they may be reassembled in the same position in the final setting up. No interchange of reamed parts will be allowed.

179. **Removing Burrs.**—The burrs on all reamed holes shall be removed by a tool countersinking about $\frac{1}{16}$ in.

Timber.

180. **Timber.**—The timber shall be strictly first-class spruce, white pine, Douglas fir, Southern yellow pine, or white oak timber; sawed true and out of wind, full size, free from wind shakes, large or loose knots, decayed or sapwood, wormholes or other defects impairing its strength or durability.

Painting

181. **Painting.**—All steel work before leaving the shop shall be thoroughly cleaned from all loose scale and rust, and be given one good coating of pure boiled linseed oil or paint as specified, well worked into all joints and open spaces.

182. In riveted work, the surfaces coming in contact shall each be painted (with paint) before being riveted together.

183. Pieces and parts which are not accessible for painting after erection shall have two coats of paint.

184. The paint shall be a good quality of red lead or graphite paint, ground with pure linseed oil, or such paint as may be specified in the contract.

185. After the structure is erected the iron work shall be thoroughly and evenly painted with two additional coats of paint, mixed with pure

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linseed oil, of such quality and color as may be selected. Painting shall be done only when the surface of the metal is perfectly dry. No painting shall be done in wet or freezing weather unless special precautions are taken. The two field coats of paint shall be of different colors.

186. Machine finished surfaces shall be coated with white lead and tallow before shipment or before being put out into the open air.

Inspection and Testing at Mill and the Shops.

187. The manufacturer shall furnish all facilities for inspecting and testing weight and the quality of workmanship at the mill or shop where material is fabricated. He shall furnish a suitable testing machine for testing full-sized members if required.

188. **Mill Orders.**—The engineer shall be furnished with complete copies of mill orders, and no materials shall be ordered nor any work done before he has been notified as to where the orders have been placed so that he may arrange for the inspection.

189. **Shop Plans.**—The engineer shall be furnished with approved complete shop plans, and must be notified well in advance of the start of the work in the shop in order that he may have an inspector on hand to inspect the material and workmanship.

190 **Shipping Invoices.**—Complete copies of shipping invoices shall be furnished the engineer with each shipment.

191. The engineer's inspector shall have full access, at all times, to all parts of the mill or shop where material under his inspection is being fabricated.

192. The inspector shall stamp each piece accepted with a private mark. Any piece not so marked may be rejected at any time, and at any stage of the work. If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the engineer.

193. **Full Size Tests.**—Full size tests of any finished member shall be tested at the manufacturer's expense, and shall be paid for by the purchaser at the contract price less the scrap value, if the tests are satisfactory. If the tests are not satisfactory the material will not be paid for and the members represented by the tested member may be rejected.

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Erection.

194. **Tools.**—The contractor shall furnish at his own expense all necessary tools, staging and material of every description required for the erection of the work, and shall remove the same when the work is completed.

195. **Risks.**—The contractor shall assume all risks from storms or accidents, unless caused by the negligence of the owner, and all damage to adjoining property and to persons until the work is completed and accepted.

196. The contractor shall comply with all ordinances or regulations appertaining to the work.

197. The erection shall be carried forward with diligence and shall be completed promptly.

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PART II. STEEL HEAD FRAMES AND COAL TIPPLES, WASHERS AND BREAKERS.

GENERAL DESCRIPTION.

198. **Types of Structure.**—The structure shall be of a type that will give maximum rigidity and strength. The structure shall be of a type in which the stresses can be calculated either by statics or by taking into account the deformations of the members (see § 230).

199. **Bracing.**—All bracing shall be stiff, and shall be riveted together at all intersections to give maximum rigidity.

200. **Proposals.**—Contractors in submitting proposals shall furnish complete stress sheets, general plans of the proposed structures, giving sizes of material, and such detail plans as will clearly show the dimensions of the parts, modes of construction and sectional areas.

201. **Detail Plans.**—The successful contractor shall furnish all working drawings required by the engineer free of cost. Working drawings will, as far as possible, be made on standard size sheets 24″ × 36″ out to out, 22″ × 34″ inside the inner border lines.

202. **Approval of Plans.**—No work shall be commenced or materials ordered until the working drawings are approved in writing by the engineer. The contractor shall be responsible for dimensions and details on the working plans, and the approval of the detail plans by the engineer will not relieve the contractor of this responsibility.

LOADS.

203. The structures shall be designed to carry the following loads without exceeding the permissible unit stresses.

204. **Dead Loads.**—The dead loads shall consist of the weight of the head sheaves, sheaves, blocks and girders, the weight of the structure, and all concentrated machinery and equipment loads.

205. **Working Loads.**—The working loads on head frames for vertical shafts shall be taken as equal to

$$K = 2W + R + (W + R)f \qquad (1)$$

where K = the working stress in lbs. at the head sheave at the instant

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of picking up the load; W = the gross load of the cage or skip and the load of ore or coal in lbs.; R = the weight of the rope from the head sheaves to the bottom of the shaft in lbs.; and f = coefficient of friction of the rope, skip and sheaves, which may be taken at 0.01 to 0.02 for vertical shafts and 0.02 to 0.04 for inclined shafts with ropes supported on rollers.

206. For inclined shafts the working load shall be taken as

$$K' = (2W + R) \sin\theta + f(W + R) \cos\theta \qquad (2)$$

where θ = the angle of inclination of the shaft with the horizontal.

207. **Breaking Load.**—The head frame shall be designed for a load in one or all of the hoisting ropes equal to the breaking stress of the hoisting rope as given in the manufacturer's catalog.

208. **Machinery Loads.**—The stresses due to machinery, crushers, tipple equipment, etc., shall be considered the same as the stresses due the working or live load.

209. **Wind Loads.**—Where the head frame or tipple is enclosed the wind load shall be assumed as 30 lbs. per sq. ft. of exposed surface acting horizontally. Where the framework is open the wind load shall be taken as 50 lbs. per sq. ft. acting on the projection of the members of the head frame or tipple. In calculating the stresses due to wind, the wind loads may be assumed as applied at the joints of the structure. Where one side of the structure is open so that a deep cup or pocket is formed the wind load shall be taken as not less than 60 lbs. per sq. ft. on the projection of the cup-like surface.

210. **Snow Loads.**—Snow loads shall be taken the same as for steel frame buildings.

Allowable Unit Stresses.

211. Steel head frames, coal tipples, coal washers and breakers, and similar structures shall be designed for the following allowable stresses.

212. **Dead Load Stresses.**—The allowable unit stresses for dead loads shall be the same as for steel frame buildings given in Part I. Snow loads shall be considered as dead loads.

213. **Working Load Stresses.**—The allowable unit stresses for working loads shall be one half the allowable unit stresses for dead load stresses as given in Part I.

214. **Bins.**—Bins shall be designed for two thirds the allowable unit stresses for dead load stresses as given in Part I.

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215. **Breaking Load Stresses.**—The allowable unit stresses for the maximum stresses due to breaking one or all the hoisting ropes shall be equal to the allowable unit stresses for dead load stresses, plus 50 per cent, equal to three times the allowable unit stresses for working loads. The breaking loads and working loads for any shaft compartment or machine will not be assumed as acting together.

216. **Machinery Load Stresses.**—The allowable unit stresses for the maximum stresses due to machinery and moving loads shall be the same as the allowable unit stresses for working loads, equal to one half the allowable unit stresses for dead load stresses.

217. **Wind Load Stresses.**—The allowable unit stresses when the wind load stress is combined with the dead load stress plus twice the working load and machinery load stresses shall not exceed the allowable unit stresses for dead loads by more than 25 per cent. If the sum of the wind load unit stress, the dead load unit stress, and twice the working load and machinery load unit stresses exceed the allowable unit stress for dead loads by more than 25 per cent the area of the section shall be increased to reduce the actual stresses to within the prescribed limit. Wind load stresses need not be combined with breaking load stresses.

218. **Reversal of Stress.**—Members subject to a reversal of stress due to a combination of dead load stresses and working load stresses shall be designed to take both tension and compression, each stress being increased by one half the smaller of the two stresses. Members subject to a reversal of stress due to wind stress combined with dead load stresses and working load stresses, or breaking load stresses combined with dead load stresses shall be designed to carry both stresses.

219. **Foundations.**—The allowable pressures for dead loads on foundations and on masonry shall be those given in § 28, § 29 and § 30. All working load, breaking load, etc., pressures shall be reduced to equivalent dead load pressures as in § 213 to § 218, inclusive.

Equipment.

220. **Skips and Cages.**—Skips and cages shall be made of structural steel, as shown on the detail drawings. They shall be provided with guide shoes and safety devices. For inclined shafts the wheels shall have phosphor bronze bushings. All skips and cages shall be provided with effective detaching hooks. The case shall be designed to take the stress due to a loaded cage or skip dropping a vertical distance of two feet.

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221. **Bin Gates.**—Unless otherwise specified all bin gates shall be of the undercut type. All gates shall be equipped with operating mechanism so that they can be opened in service by one man.

222. **Screens.**—Fixed screens shall be made of bars as shown on the drawings and shall be supported so that the bars will not be permanently deflected under the load. The screen bars shall be placed at an angle so that they will screen the ore or coal without choking up.

223. Shaking screens shall be carried on rollers and be driven by eccentric connecting bars. They shall be placed at proper slopes, and shall be provided with all necessary gates. Unless otherwise specified the screens shall be made of structural steel.

224. Rotary screens shall be made of structural and machinery steel, and shall perform the work required by the specifications.

225. **Coal Tipples or Dumps.**—Coal tipples or dumps shall be provided as shown on the detail plans or called for in the specifications.

226. **Dumping Devices.**—Where self-dumping skips or cages are used an efficient and satisfactory dumping device shall be provided.

227. **Head Sheaves.**—The head sheaves shall be substantial with the top flanges turned smooth and true to receive the hoisting rope. The sheave wheel shaft shall be of the best grade of machinery steel of ample strength, carefully and truly made. The sheave boxes shall be lined with the best quality of anti-friction metal and shall be adjustable to take up the wear. Unless otherwise specified the sheave wheels shall have wrought iron spokes.

228. **Landing Stage.**—An efficient landing device shall be furnished.

Details of Construction.

229. Unless otherwise provided for the details of construction are to be the same as for steel frame buildings.

230. **Design.**—In designing head frames, coal tipples, coal washers and breakers and similar structures care shall be used to strongly brace the different parts of the structure in order that it may be rigid. Preference shall be given to types of structures that are statically determinate. Where 4-post head frames and other statically indeterminate structures are used the stresses shall be calculated by taking account of the deformation and distortions of the members. All bracing is to be made of stiff members; the use of rods or bars will not be per-

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mitted, except for sag rods and anchors. It is very important that head frames, coal tipples, coal washers and breakers and similar structures be made very rigid.

231. **Lengths of Compression Members.**—The length of compression members in head frames and shaker structures shall not exceed 100 times the least radius of gyration for main members nor 140 times the least radius of gyration for secondary bracing.

232. **Lengths of Tension Members.**—The length of tension members in head frames, shaker structures and supports for moving machinery shall not exceed 150 times the least radius of gyration for main members, nor 200 times the least radius of gyration for secondary bracing. The length of a tension member is to be taken as the distance center to center of end connections.

233. **Splices.**—All splices in main members shall be designed to carry the full strength of the member.

234. **Reaming.**—The rivet holes for all field splices in head frames, shaker structures and supports of moving machinery shall be punched to a diameter $\frac{3}{16}$ in. less than the finished hole and shall be reamed to the required size with the members bolted in place with an iron templet. All metal more than $\frac{5}{8}$ in. thick shall be punched and reamed, or be drilled from the solid.

235. **Minimum Thickness of Metal.**—The minimum thickness of metal in plates and sections shall be $\frac{5}{16}$ in., except for fillers.

236. **Erection.**—All field connections shall be riveted. Before the riveting is begun all field connections shall be fully drawn up with field bolts, in not less than one half the holes of each joint.

237. **Materials and Workmanship.**—All materials and workmanship shall comply with the Specifications for Steel Frame Buildings unless otherwise specified.

238. **Painting.**—All steel work shall receive one coat of satisfactory graphite or carbon paint at the shop. Before erecting all abraded spots shall be touched up, and all rivet heads shall be painted as soon as accepted by the inspector. After the erection is complete all structural steel work shall be given two coats of satisfactory graphite or carbon paint. The three coats of paint shall be of different colors.

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APPENDIX II.

SPECIFICATIONS FOR TIMBER MINE STRUCTURES.
GENERAL DESCRIPTION.

1. **General Clauses.**—Paragraphs 1 to 11, inclusive, in the "Specifications for Steel Mine Structures" shall apply to timber mine structures.

2. **Loads.**—Paragraphs 12 to 31, inclusive, in the "Specifications for Steel Mine Structures" shall apply to timber mine structures.

ALLOWABLE STRESSES.

3. **Allowable Stresses in Timber.**—The allowable stresses for dead loads on timber structures shall be taken as in Table I.

TABLE I.

ALLOWABLE WORKING UNIT STRESSES FOR DEAD LOADS IN LBS. PER SQ. IN.

Kind of Timber	Transverse Loading, S	End Bearing	Columns Under 10 Diameters, C	Bearing Across Fiber	Shear		Modulus of Elasticity, E
					Parallel to Grain	Longitudinal Shear in Beams	
White Oak	1,200	1,200	1,000	450	200	110	1,150,000
Long Leaf Yellow Pine	1,300	1,300	1,000	300	180	120	1,610,000
White Pine and Spruce	1,000	1,000	800	200	100	70	1,130,000
Western Hemlock	1,000	1,000	800	200	160	100	1,480,000
Douglas Fir	1,200	1,200	1,000	350	180	110	1,510,000

Columns may be used with a length not exceeding 45 times the least dimension. The unit stress for lengths of more than 10 times the least dimension shall be reduced by the following formula:

$$P = C - \frac{C}{100} \frac{l}{d}$$

where C = unit stress as given above for short columns; P = allowable unit stress in lbs. per sq. in.; l = length of column in in.; d = least side of column in in.

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4. **Live Loads.**—The allowable stresses for working loads, machinery loads and moving loads shall be one half ($\frac{1}{2}$) of the allowable stresses for dead loads as given in § 3.

5. **Breaking Loads.**—The allowable stresses for breaking loads in head frames shall be three halves ($\frac{3}{2}$) of the allowable stresses for dead loads as given in § 3.

6. **Wind Loads.**—When the wind load stresses are less than 25 per cent of the sum of the dead load stresses and twice the working load or machinery load stresses, the wind load stresses need not be considered. Where the wind load stresses exceed 25 per cent of the sum of the dead load stresses and twice the working load or machinery load stresses, the member shall be designed by dividing the sum of the dead load stress, twice the working load stress and the wind load stress by the allowable stress for dead loads plus 25 per cent. Wind load stresses need not be considered in combination with breaking load stresses.

7. **Allowable Stresses in Steel and Iron.**—The allowable unit stresses in structural steel and iron shall be as given in the "Specifications for Steel Mine Structures."

MATERIALS. STRUCTURAL TIMBER.

8. **Kinds of Timber.**—For structures carrying live or moving loads timber shall preferably be long leaf yellow pine, Douglas fir, white oak, white pine, or Western hemlock; for other purposes short leaf yellow pine, spruce, or other equivalent good timber may be used.

9. **General Requirements.**—All timber must be cut within 18 months of the time of delivery from sound live trees and be sawed to standard size. It must be close grained and solid, free from defects, such as injurious ring shakes and cross grain, unsound or loose knots, knots in groups, large pitch pockets, decay, or other defects that will materially impair its strength or fitness for the purpose intended.

10. **Size of Sawed Timber.**—All timber shall be sawed true and out of wind and shall, when dry, not measure scant in thickness more than the following:

Flooring and boards, rough size, up to $1\frac{1}{2}$ in. thick, may be scant $\frac{1}{16}$ in.

Planks and timbers, rough size, from $1\frac{3}{4}$ to $5\frac{3}{4}$ in. thick, may be scant $\frac{1}{8}$ in.

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Timbers, rough size, 6 in. thick and up, may be scant $\frac{1}{4}$ in. For example, a 12 in. × 12 in. timber may be $11\frac{3}{4}$ in. × $11\frac{3}{4}$ in.

11. **Size of Dressed Timber.**—When dressed timber more than $1\frac{1}{2}$ in. in thickness is required, a reduction of $\frac{1}{8}$ in. for each surface planed will be permitted in addition to the reduction that is allowed in § 10. For example a 12 in. × 12 in. timber S.4 S. may be $11\frac{1}{2}$ in. × $11\frac{1}{2}$ in.

12. **Dimension Timber.**—Dimension timber when used for beams, stringers, caps, posts, and sills shall show not less than seventy-five (75) per cent heart on each of the four faces, measured across the sides anywhere in the length of the piece. There shall be no loose knots, or knots greater than $2\frac{1}{2}$ in. or one quarter ($\frac{1}{4}$) the width of the face of the stick in which they occur. Knots must not be located in groups and no knot must be nearer the edge of the stick than one quarter ($\frac{1}{4}$) the width of the face.

When used for other purposes dimension timber shall be square-edged, with the exception of 1 in. wane on one edge or $\frac{1}{2}$ in. wane on two edges, and ring shakes must not extend over one eighth ($\frac{1}{8}$) the length of the piece.

13. **Timbers for Head Frames.**—Timbers for back braces and main columns and for main timbers in head frames shall show not less than ninety (90) per cent heart on each side and edge, measured anywhere in the length of the piece; shall be out of wind and free from shakes, splits or pitch pockets over one eighth ($\frac{1}{8}$) in. wide or three (3) in. long. Knots shall not be more than one and one half ($1\frac{1}{2}$) in. in diameter, nor closer together on each surface than one in any four (4) linear feet, but if knots are one (1) in. in diameter or less, one in every three (3) feet may be allowed.

14. **Planks and Scantling.**—Planks and scantling, when used for floors, rafters, purlins, girts, struts and bracing, shall be square edged, shall show one face all heart, the other face and two sides shall show not less than seventy-five (75) per cent heart, measured across the face or sides anywhere in the length of the piece; and shall be free from loose knots, or sound knots more than $1\frac{1}{2}$ in. in diameter.

When used for other purposes planks and scantling shall be square-edged, free from knots more than 2 in. in diameter, and wane extending over 5 per cent of the surface area may be permitted in 5 per cent of the total number of pieces of any size.

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15. **Piles.**—Timber piles shall preferably be of long leaf yellow pine, white oak, Douglas fir, cedar or cypress.

Piles shall be cut from sound, live trees, shall be straight, close-grained and solid, free from defects such as injurious ring shakes, large and unsound or loose knots, decay or other defects that will materially impair the strength or durability. The diameter of round piles near the butt shall not be less than 14 in. nor more than 18 in., and at the tip of piles under thirty (30) ft. in length not less than 8 in. nor less than 6 in. for piles more than thirty (30) ft. long. Piles must be cut above the ground-swell and must taper evenly from butt to tip. Short bends will not be allowed. A line drawn from the butt to the tip shall lie within the body of the pile. All piles shall be cut square at their ends and shall be stripped of their bark.

16. **Flooring.**—Flooring shall preferably be of yellow pine or maple, furnished in two grades (select and common) usually in lengths 4 to 16 ft., and not over $2\frac{1}{2}$ in. face. The thickness of flooring shall be understood as the thickness after dressing.

(a) *Select Flooring.*—Select flooring shall be edge grain, kiln dried, matched, tongued and grooved, well manufactured so as to be free from planer marks, edge splinters, grain slivers, etc. It shall show one face all heart, and shall be free from knots, shakes, sap and pitch pockets.

(b) *Common Flooring.*—Common flooring shall be select flooring except that the material may show one solid knot not more than one (1) in. in diameter every three (3) feet and unimportant pitch streaks and sap stains.

17. **Ceiling.**—Ceiling shall be graded as flooring, but shall in addition be double beaded and be very carefully dressed.

18. **Shingles.**—Shingles shall be of pine, cedar, or cypress, as specified in the contract. They shall be from 16 to 18 in. in length, from 4 to 6 in. wide, $\frac{1}{16}$ in. thick at the tip and $\frac{7}{16}$ to $\frac{1}{2}$ in. thick at the butt. The first 10 in. from the butt shall be absolutely free from sap, knots or other defects.

STRUCTURAL STEEL AND IRON AND METAL DETAILS.

19. **Structural Steel and Iron.**—Structural steel and wrought and cast iron shall comply with the specifications given in the "Specifications for the Design of Steel Mine Structures."

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20. **Bolts.**—Bolts shall be of wrought iron or steel, made with square heads, standard size, the length of thread to be 2½ times the diameter of bolt. The nuts shall be made square, standard size, with thread fitting closely the thread of the bolt. Threads shall be cut according to U. S. standards.

21. **Drift Bolts.**—Drift bolts shall be of wrought iron or steel, with or without square head, pointed or without point, as may be called for on the plans.

22. **Spikes.**—Spikes shall be of wrought iron or steel, square or round, as called for on the plans; steel wire spikes, when used for spiking planking, shall not be used in lengths more than 6 in.; if greater lengths are required, wrought or steel spikes shall be used.

23. **Cast Washers.**—Cast washers shall be of cast iron. The diameter shall be not less than 3½ times the diameter of bolt for which it is used, and its thickness equal to the diameter of bolt; the diameter of hole shall be ⅛ in. larger than the diameter of the bolt.

24. **Wrought Washers.**—Wrought washers shall be of wrought iron or steel. The diameter shall be not less than 3½ times the diameter of bolt for which it is used, and not less than ¼ in. thick. The hole shall be ⅛ in. larger than the diameter of the bolt.

25. **Special Castings.**—Special castings shall be made true to pattern and out of wind, free from flaws and excessive shrinkage; size and shape to be as called for by the plans.

DETAILS OF DESIGN AND CONSTRUCTION.

26. **Workmanship.**—The workmanship shall be of the best quality in each class of work. Details, fastenings, connections and finish shall be of the best method of construction in general use on first-class work.

27. **Holes.**—Holes shall be bored for all bolts. The depth of the hole and the diameter of the auger for drift bolts and dowels shall be specified by the engineer.

28. **Framing.**—Framing shall be accurately fitted; no blocking or shimming will be allowed in making joints. Timbers shall be cut off with the saw; no axe to be used.

29. **Joints and points** of bearing for which no fastening is shown shall be fastened as specified by the engineer.

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30. **Piles.**—Piles shall be driven to a firm bearing, or until six blows of a hammer weighing 2,000 lbs., falling freely 20 ft. (or a hammer and fall producing same effect) are required to give an average penetration of one half ($\frac{1}{2}$) in. per blow, except in soft ground where the safe load will be calculated by the formula

$$P=\frac{2W\cdot h}{s+1}$$

Where P = safe load on pile in tons; W = weight of hammer in tons; h = free fall of hammer in ft.; s = average penetration for last six blows in in. For a steam hammer use $\frac{1}{10}$ in place of unity in second member of the formula.

Piles shall have a penetration of not less than 10 ft. in hard material, such as gravel, and not less than 15 ft. in loam or soft material.

Piles shall be sawed off to one plane and shall be trimmed so as not to leave any horizontal projection outside the cap.

31. **Joists and Flooring.**—Joists and flooring, carrying plastering shall be designed so that the maximum deflection shall not exceed $1/360$ the span.

32. **Bracing and Struts.**—All bracing and struts shall be properly framed and securely fastened with bolts or spikes as specified.

33. **Painting.**—All dressed timber framework and the exterior of buildings and other structures, and all metal work shall be painted with two coats of good paint by mixing (as specified) lbs. of iron ore, white lead, red lead or other pigment as specified with each gallon of pure linseed oil. All surfaces shall be dry and clean when the paint is applied, and no painting shall be done in wet or freezing weather.

34. **Erection.**—The contractor shall furnish all necessary tools, machinery, labor, temporary staging and outfit required. After the work is completed he shall remove all refuse and materials and leave the structure in a finished condition.

The contractor shall be liable for all damages to property or persons due to his operations. He shall procure at his own expense all necessary permits and shall comply with all municipal ordinances.

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APPENDIX III.

REINFORCED CONCRETE STRUCTURES.

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CHAPTER I.

Data for the Design of Reinforced Concrete Structures.

(1) **Materials.**—The materials and workmanship for reinforced concrete shall meet the requirements of the "Specifications for Plain and Reinforced Concrete," given in Chapter III.

The concrete recommended for general use is a mixture of one part Portland cement, two parts of fine aggregate, and four parts of coarse aggregate. A mixture of one part of Portland cement, three parts of fine aggregate, and six parts of coarse aggregate may be used in monolithic walls and foundations.

(2) **Dimensions.**—The span length for beams and slabs is to be taken as the distance from center to center of the supports, but is not to exceed a clear span plus the depth of beam or slab.

(3) **Internal Stresses.**—The internal stresses are to be calculated upon the basis of the following assumptions:

(*a*) A plane section before bending remains plane after bending.

(*b*) The distribution of compressive stresses in members subject to bending is rectilinear.

(*c*) The ratio of the moduli of elasticity of steel and concrete is $n=15$.

(*d*) The tensile stresses in the concrete are neglected in calculating the moment of resistance of beams.

(*e*) The initial stress in the reinforcement due to contraction and expansion in the concrete is neglected.

(*f*) The depth of a beam is to be taken as the distance from the compressive face to the centroid of the tension reinforcement.

(*g*) The effective depth of a beam at any section is the distance from the centroid of the compressive stresses to the centroid of the tension reinforcement ($j.d$).

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(*h*) Perfect adhesion is assumed between concrete and reinforcement. In compressive stress the two materials are therefore stressed in proportion to their moduli of elasticity.

(*i*) The maximum shearing unit stress in beams is to be taken as the total shear at the section divided by the product of the width of the section and the effective depth of the section considered ($v = V \div b.j.d$). This maximum shearing unit stress is to be used in place of the diagonal tension stress in calculations for web stresses.

(*j*) The bond unit stress is equal to the vertical shear divided by the product of the total perimeter of the reinforcement in the tension side of the beam and the effective depth at the section considered ($u = V \div j.d.\Sigma o$).

(*k*) In concrete columns, walls and slabs, concrete to a depth of $1\frac{1}{2}$ inches and in floor slabs 1 inch is to be considered as a protective covering, and is not to be included in the effective section.

(4) **Web Stresses.**—When the maximum shearing stresses exceed the value allowed for the concrete alone, web reinforcement must be provided to aid in carrying the diagonal tensile stresses. This web reinforcement may consist of bent-up bars or inclined or vertical members, attached to or hooked around the horizontal reinforcement. Where inclined members are used, the connection to the horizontal reinforcement shall be such as to insure against slip.

In the calculation of web reinforcement when the concrete alone is insufficient to take the diagonal tension, the concrete may be assumed to carry a shear of 40 pounds per square inch. The remainder of the shear is to be carried by metal reinforcement consisting of bent-up bars or stirrups, but preferably both. The requisite amount of such reinforcement may be estimated on the assumption that the entire shear on a section, less the amount assumed to be carried by the concrete, is carried by the reinforcement.

(5) **Working Stresses.**—The following working stresses are to be used with concrete of such a quality as to be capable of developing an average compressive strength of at least 2,000 pounds per square inch,

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when tested in cylinders 6 inches in diameter and 6 inches long, after having been stored one day in air and 27 days in water, under laboratory conditions of manufacture and storage, the mixture being of the same consistency as is used in the field.

Allowable stress in pounds per square inch.

Structural steel in tension	16,000
Steel in compression, fifteen times the compressive stresses in the surrounding concrete.	
Concrete in bearing, where the total surface is at least twice the bearing area	650
Concrete in direct compression without reinforcement, on a length not exceeding six times the least width	450
Columns in which the unsupported length is not more than fifteen times the least width are to be designed for the following stresses:	
(*a*) Columns with not less than 1 nor more than 4 per cent of longitudinal reinforcement only	450
(*b*) Columns reinforced with bands or hoops spaced not less than 1/7 the diameter of the column. The total amount of reinforcement being not less than 1 per cent of the inclosed column	540
(*c*) Columns reinforced with not less than 1 per cent nor more than 4 per cent of longitudinal bars and with bands or hoops as in (*b*)......................	650
(*d*) Columns reinforced with structural steel column-units that thoroughly incase the concrete core	650
Concrete in compression, on extreme fiber in cross-bending ...	650
Concrete in shear not combined with tension or compression in concrete	120
Concrete in shear, where the shearing stress is used as the measure of web stress	40
Concrete in shear where part of the rods are bent up to take diagonal tension	60

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Concrete in shear where the beam is completely reinforced for shear and diagonal tension	120
Bond stress for plain round or square bars	80
Bond stress for plain round or square bars with hooked ends bent 180° around a diameter of 3 diameters of the bar and with a short length of bar extending beyond the bend	100
Bond stress for drawn wire	40
Bond stress for deformed bars depending upon form	80–150

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CHAPTER II.

Formulas for the Design of Reinforced Concrete Structures.

The following formulas are based upon the assumptions and principles given in Chapter I.

STANDARD NOTATION. 1. Rectangular Beams.

f_s = tensile unit stress in steel.

f_c = compressive unit stress in concrete.

ϵ_s = elongation of steel due to f_s.

ϵ_c = shortening of concrete due to f_c.

E_s = modulus of elasticity of steel.

E_c = modulus of elasticity of concrete.

$$n = \frac{E_s}{E_c}.$$

M_s = moment of resistance relative to the steel.

M_c = moment of resistance relative to the concrete.

M = moment of resistance, or bending moment in general.

A = steel area.

T = total tension.

C = total compression.

b = breadth of beam.

d = depth of beam to center of steel.

k = ratio of depth of neutral axis to depth d.

z = depth of resultant compression below top of beam.

j = ratio of lever arm to resisting couple to depth d.

$j.d = d - z$ = arm of resisting couple.

$p = \frac{A}{b.d}$ = steel ratio (not percentage).

$R_s = f_s.p.j$ = coefficient of strength relative to steel.

$R_c = \frac{1}{2} f_c.k.j$ = coefficient of strength relative to concrete.

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2. **T-Beams.**

b = width of flange.
b' = width of stem or web.
t = thickness of flange.

3. **Beams Reinforced for Compression.**

A' = area of compressive steel.
p' = steel ratio for compressive steel.
f_s' = compressive unit stress in steel.
C' = total compressive stress in steel.
d' = depth to center of compressive steel.
z = depth to resultant of C and C'.

4. **Shear and Bond.**

V = total shear.
v = maximum shearing unit stress $= V/bjd$.
v' = average shearing unit stress $= V/bd$.
u = bond stress per unit area of bar.
o = circumference or perimeter of bar.
Σo = sum of perimeters of bars.

5. **Columns.**

A = total net area of column.
A_s = area of longitudinal steel.
A_c = area of concrete.
$p = \dfrac{A_s}{A}$ = steel ratio for longitudinal steel.
p' = steel ratio of the hoops of hooped columns.
P = strength of plain concrete column.
P' = strength of reinforced column.
f = average unit stress for entire cross-section.

STRESSES IN RECTANGULAR BEAMS.—In (c), Fig. 1, b is the breadth, d is the depth of the beam above the center of the reinforcing steel, $k.d$ is the distance of the neutral axis below the top of the beam, and $j.d$ is the arm of the resisting couple, k and j being ratios.

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In (a) the deformations are shown to be proportional to the distances from the neutral axis, and in (b) the stress in the steel is n times the stress in the concrete at the same distance from the neutral axis.

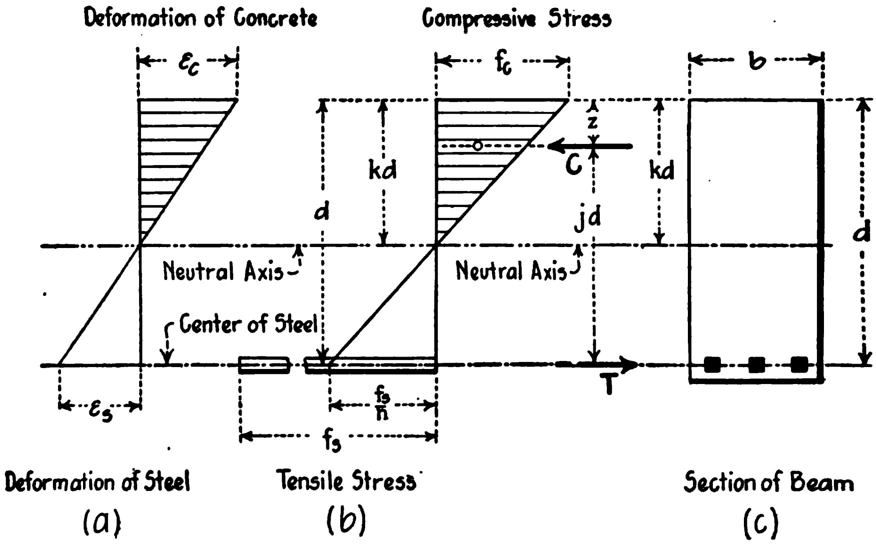

FIG. 1.

Neutral Axis and Arm of Resisting Couple.—Now the sum of the horizontal compressive stresses is equal to the horizontal tensile stress, and

$$\tfrac{1}{2}f_c.b.k.d = f_s.A \tag{1}$$

Substituting the value of $A = p.b.d$, and reducing

$$\tfrac{1}{2}f_c.k = f_s.p \tag{2}$$

From (b) Fig. 1, we have

$$f_c : f_s/n :: k.d : d(1-k),$$

and

$$f_s.k.d = f_c.n.d(1-k)$$

$$f_s.k = f_c.n(1-k) \tag{3}$$

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Substituting the value of f_s in (2) in (3)

$$\tfrac{1}{2}f_c.k^2 = f_c.p.n(1-k),$$

and

$$k^2 = 2p.n(1-k)$$

$$k = \sqrt{2p.n + p^2.n^2} - p.n \qquad (4)$$

This formula shows that k is a constant for all beams having a given percentage of reinforcement and the same grade of concrete. The values of k for $n=15$ and for different values of p are given in the upper part of Fig. 2.

The centroid of the compressive stresses is $z=\frac{1}{3}k.d$ below the top of the beam, and the arm of the resisting couple is

$$j.d = d - \tfrac{1}{3}k.d, \text{ or } j = 1 - \tfrac{1}{3}k \qquad (5)$$

Values of j for $n=15$ and for different values of p are given in Fig. 2. It will be seen that for $f_s=15{,}000$ to $16{,}000$ lbs. per sq. in. and $f_c=600$ to 650 lbs. per sq. in., j may be taken as $\frac{7}{8}$.

Moment of Resistance.—If the beam is under-reinforced its strength will depend on the steel, and

$$M_s = T.j.d = f_s.A.j.d = f_s.p.j.b.d^2 \qquad (6)$$

If the beam is over-reinforced its strength will depend on the concrete, and

$$M_c = C.j.d = \tfrac{1}{2}f_c.b.k.d.j.d = \tfrac{1}{2}f_c.k.j.b.d^2 \qquad (7)$$

The resisting moment of the beam is the smaller of the two values of M.

Now if $R_s = f_s.p.j$, and $R_c = \frac{1}{2}f_c.k.j$, equations (6) and (7) become

$$M_s = R_s.b.d^2 \qquad (6a)$$

$$M_c = R_c.b.d^2 \qquad (7a)$$

$$d = \sqrt{\frac{M}{R.b}} \qquad (6b)$$

Fiber Stresses.—To calculate the unit fiber stresses for a given

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bending moment solve equations (6) and (7), and

$$f_s = \frac{T}{A} = \frac{M}{A.j.d} = \frac{M}{p.j.b.d^2} \tag{8}$$

$$f_c = \frac{2M}{b.k.j.d^2} = \frac{2f_s.p}{k} \tag{9}$$

Steel Ratio.—If k be eliminated by solving equations (2) and (3) the steel ratio will be

$$p = \frac{\frac{1}{2}}{\frac{f_s}{f_c}\left(\frac{f_s}{n.f_c} + 1\right)} \tag{10}$$

If a value of p less than that given by (10) is used the steel determines the strength of the beam, while if p is greater the concrete will determine the strength of the beam.

Diagram for Rectangular Beams.—In Fig. 2 are given values of k and j for $n = 15$ and for different values of p. Values of $R_c = \frac{M}{b.d^2}$ are given for different values of f_c and p, and values of $R_s = \frac{M}{b.d^2}$ are given for different values of f_s and p. The use of the table will be shown by three problems.

Problem 1. Moment of Resistance.—Given the following data: $b = 10''$, $d = 20''$, $f_s = 16{,}000$ lbs., $f_c = 600$ lbs., 2 steel bars 1″ □ ($p = 0.01$), find M_s and M_c.

Solution.—In Fig. 2 find the value of percentage of reinforcement $p = 1$ per cent, on lower margin and follow the vertical line to curved line $f_c = 600$, then follow to the left on a horizontal line and find $R_c = 107$ on left margin. In like manner $R_s = 138$, which will overstress the concrete. The resisting moment will then be $M = R_c.b.d^2 = 107 \times 10 \times 20^2 = 428{,}000$ in.-lbs.

Problem 2. Fiber Stresses.—Given the following data: $b = 10''$, $d = 20''$, $p = 0.009$ (0.9 per cent), $M = 360{,}000$ in.-lbs., find f_s and f_c.

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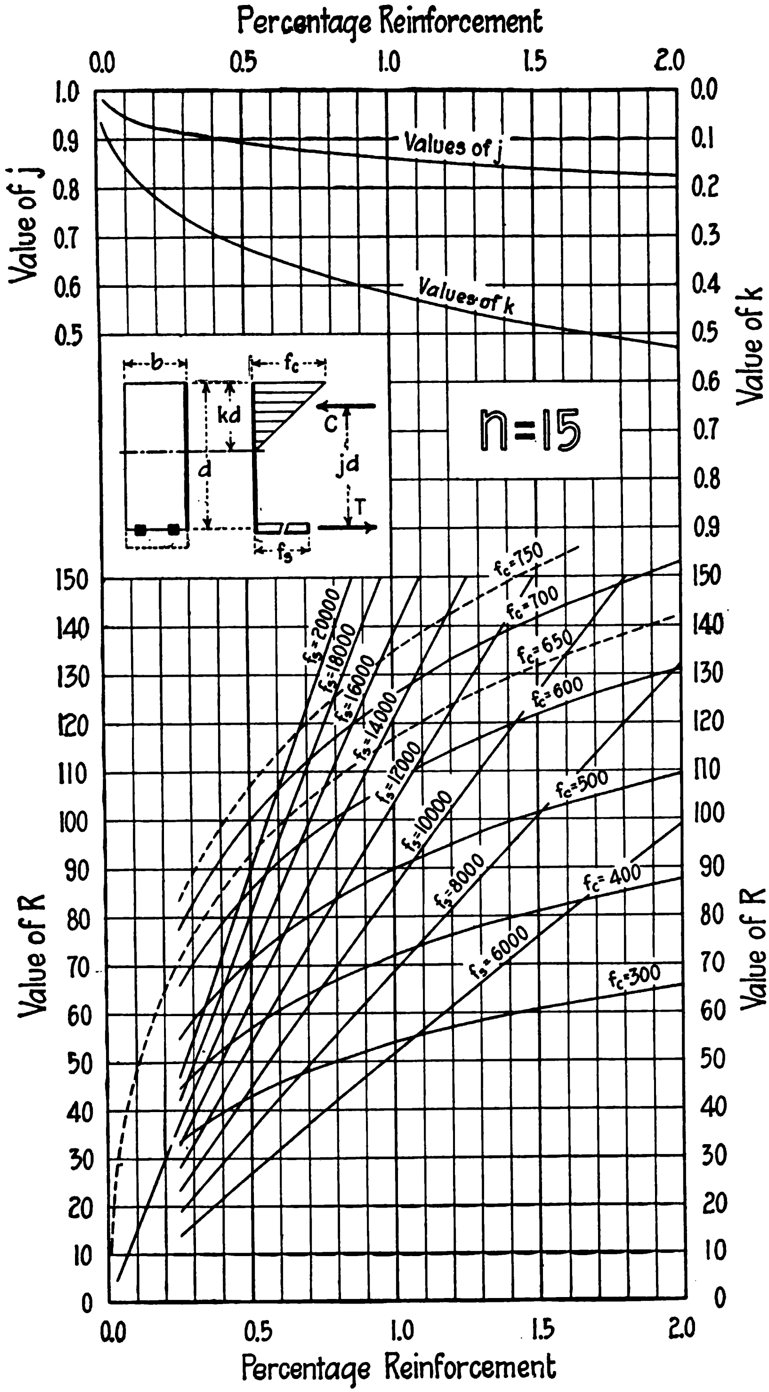

FIG. 2.

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Solution.—$R = M/b.d^2 = 90$. In Fig. 2 the intersection of a vertical line through reinforcement = 0.9 per cent and a horizontal line through $R = 90$, gives $f_c = 520$ and $f_s = 11{,}700$.

Problem 3. Cross-section of Beam and Percentage of Reinforcement.—Given $M = 360{,}000$ in.-lbs., $f_s = 14{,}000$, $f_c = 500$ lbs., to find b, d and p.

Solution.—In Fig. 2 the intersection of curved line $f_c = 600$ and straight line $f_s = 14{,}000$ gives on the lower margin, $p = 0.0084$ (0.84 per cent), and on left margin gives $R = 102$. Then $b.d^2 = M/R = 3{,}530$. Now if $b = 10''$, then $d = 19''$.

STRESSES IN T-BEAMS.—There will be two cases: (1) when the neutral axis is in the flange, and (2) when the neutral axis is in the web.

Case I. The Neutral Axis in the Flange.—The formulas for a rectangular beam apply where b is the flange width and $p = A \div b.d$, not $A \div b'.d$.

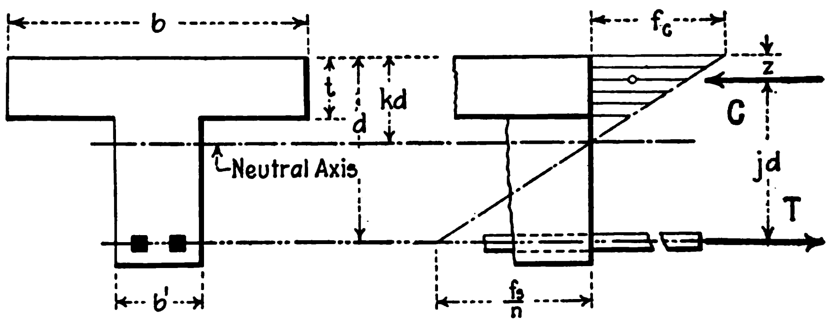

FIG. 3.

Approximate Formulas.—It will be seen in Fig. 3 that $j.d$ is always greater than $d - t/3$, and the following formulas are on the safe side. $M_s = f_s.A(d - t/3)$, and $f_s = M_s/A(d - t/3)$. There is no corresponding approximate formula for the concrete.

Case II. The Neutral Axis in the Web.—Where the thickness of the flange t is large as compared with the depth of the beam, or as com-

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pared with the width of the web, the compression in the web may be neglected.

(1) **The Compression in the Web Neglected. Neutral Axis and Arm of Resisting Couple.**—As in the rectangular beam

$$f_s.k = f_c.n(1 - k) \qquad (3)$$

and

$$k = \frac{1}{1 + f_s/n.f_c} \qquad (11)$$

The average unit compressive stress in the flange is

$$\tfrac{1}{2}[f_c + f_c(1 - t/k.d)] = f_c\left(1 - \frac{t}{2k.d}\right),$$

and the total compression is

$$C = f_c\left(1 - \frac{t}{2k.d}\right)b.t$$

Now since $C = T$

$$f_s.A = f_c\left(1 - \frac{t}{2k.d}\right)b.t \qquad (12)$$

Solving (3) and (12) for k we have

$$k = \frac{2n.d.A + b.t^2}{2n.d.A + 2b.d.t} \qquad (13)$$

Substituting $p.b.d$ for A, we have

$$k = \frac{2n.p.d^2 + t^2}{2n.p.d^2 + 2d.t} \qquad (14)$$

The arm of the resisting couple is $d - z$, where z is the distance from the top of the beam to the center of the shaded area in Fig. 3.

$$z = \frac{3k.d - 2t}{2k.d - t} \times \frac{t}{3} \qquad (15)$$

Also

$$j.d = d - z \qquad (16)$$

Substituting k from (14) in (15) and z from (15) in (16) we have

$$j = \frac{6 - 6t/d + 2(t/d)^2 + (t/d)^3/2p.n}{6 - 3t/d} \qquad (17)$$

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Moment of Resistance.—If the beam is under-reinforced the strength of the beam will depend upon the steel, and

$$M_s = f_s.A(d - z) = f_s.A.j.d \qquad (18)$$

If the beam is over-reinforced the strength of the beam will depend upon the concrete, and

$$M_c = f_c\left(1 - \frac{t}{2k.d}\right)b.t.j.d \qquad (19)$$

Unit Stresses.—The values of f_s and f_c may be obtained from (8) and (9) respectively, or from

$$C = T = M/j.d, \text{ and } f_s = \frac{T}{A} \qquad (20)$$

$$f_c = \frac{f_s}{n} \times \frac{k}{1 - k} = \frac{f_s.p}{\left(1 - \frac{t}{2k.d}\right)\frac{t}{d}} \qquad (21)$$

2. **Compression in Web Considered.**—Where the flange is thin as compared with the depth of the beam, d, and width of web, b', it may become necessary to consider the compression in the web. In the same manner as in (1), we have

$$k.d = \sqrt{\frac{2n.d.A + (b - b')t^2}{b'} + \left(\frac{n.A + (b - b')t}{b'}\right)^2} - \frac{n.A + (b - b')t}{b'}$$

$$z = \frac{(k.d.t^2 - \frac{2}{3}t^3)b + \left[(k.d - t)^2\left(t + \frac{k.d - t}{3}\right)\right]b'}{b.t(2k.d - t) + b'(k.d - t)^2} \qquad (23)$$

$$j.d = d - z \qquad (24)$$

$$M_s = f_s A.j.d \qquad (25)$$

$$M_c = \frac{f_c}{2k.d}[(2k.d - t)b.t + (k.d - t)^2 b']j.d \qquad (26)$$

Design of T-Beams.—Where the dimensions and reinforcement of the beam are given the safe load can be calculated by the preceding formulas. If the value of $k.d$ is less than t the problem comes under Case I, and the formulas for rectangular beams may be used.

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In designing a T-beam the value of b' will be determined by the space required for the reinforcing rods and to give the required shearing strength. The thickness of the flange t will be governed by the width of flange b; standard specifications require that the overhang of the flange shall not be greater than $4t$, or that $b \leq 8t + b'$, also that the total flange width shall not exceed one-fourth the span.

STRESSES IN BEAMS REINFORCED FOR COMPRESSION.—The beam is reinforced with steel on both the compression and tension sides.

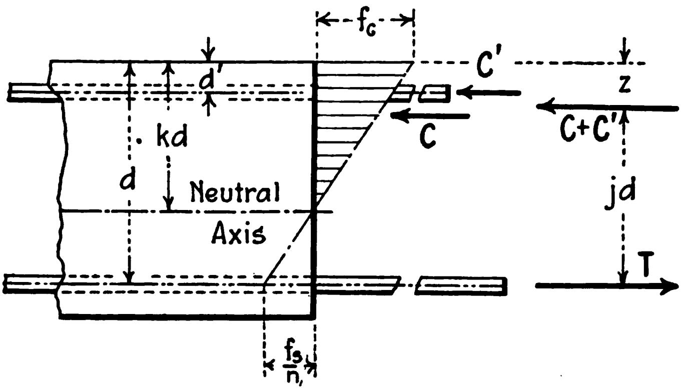

FIG. 4.

Neutral Axis and Arm of Resisting Couple.—From Fig. 4, as in Fig. 1, we have

$$f_s.k = f_c.n(1 - k) \qquad (3)$$

Also

$$f_s'.k.d = f_c.n(k.d - d'),$$

and

$$f_s' = f_c.n\left(1 - \frac{d'}{k.d}\right) \qquad (27)$$

For simple flexure $T = C + C'$, and $f_s.A = \frac{1}{2}f_c.b.k.d + f_s'.A'$ (28)

Substituting values of f_c and f_s' from (3) and (27) in (28), we have

$$k^2 + 2n(p + p')k = 2n(p + p'.d'/d)$$

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and solving for k,

$$k=\sqrt{2n(p+p'd'/d)+[n(p+p')]^2}-n(p+p') \qquad (29)$$

The arm of the resisting couple is

$$j.d=d-z \qquad (30)$$

where z is given by the equation

$$z=\frac{\frac{1}{3}k^3.d+2p'.n.d'\left(k-\frac{d'}{d}\right)}{k^2+2p'.n\left(k-\frac{d'}{d}\right)} \qquad (31)$$

Moment of Resistance.—If the beam is under-reinforced on the tension side the strength of the beam is determined by the steel, and

$$M_s=f_s.A.j.d=f_s.p.j.b.d^2 \qquad (32)$$

If the beam is over-reinforced on the tension side, the strength of the beam is determined by the compressive resistance and

$$M_c=\tfrac{1}{2}f_c.k(1-\tfrac{1}{3}k)b.d^2+f_s'.p'.b.d(d-d') \qquad (33)$$

If the value of f_s' from (27) be substituted in (33), then

$$M_c=f_c.b.d^2[k(\tfrac{1}{2}-\tfrac{1}{6}k)+n.p'(k-d'/d)(1-d'/d)/k] \qquad (34)$$

Fiber Stresses.—The stress f_s for a moment M is

$$f_s=\frac{M}{A.j.d}=\frac{M}{p.j.b.d^2} \qquad (35)$$

while the compressive stresses may be calculated by equations (3) and (27).

Approximate Formulas.—For approximate calculations assume that $k=0.45$ and $j=0.85$, and then

$$M_s=0.85p.f_s.b.d^2 \qquad (36)$$

$$M_c=(0.19+10.5p')f_c.b.d^2 \qquad (37)$$

$$f_s=1.18M/p.b.d^2 \qquad (38)$$

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FLEXURE AND DIRECT STRESS.—When a member carries direct stress and at the same time acts as a beam, there are both direct stresses and bending stresses at any section. A common example is where the resultant of the external forces on a beam acting on one side of the section is not normal to the beam. There are two cases: (1) where the neutral axis is entirely outside of the beam and the combined stresses are all tension or all compression, and (2) where the neutral axis is inside the section and the stresses on the section are both tension and compression.

The following additional notation is required:

$P=$ resultant of all external forces acting on a beam on either side of the section.

$N=$ component of P normal to section.

$e=$ eccentric distance of P.

$M=$ bending moment on section $=N.e$.

$A'=$ area of steel near face most highly stressed.

$d'=$ distance from upper face to center of steel A'.

$A=$ area of steel near other face.

$d=$ distance from upper face to center of steel A.

$h=$ height of section.

$p'=$ steel ratio $A'/b.h$.

$p=$ steel ratio $A/b.h$.

$y=$ distance from upper face to center of the transformed section.

$A_t=$ area of the transformed section.

$I_t=$ moment of inertia of transformed section with reference to its centroidal axis.

$I_c=$ moment of inertia of the concrete with reference to the same axis.

$I_s=$ moment of inertia of the steel with reference to the same axis.

Case I. Stresses all Compression.—(*a*) The unit stresses in the concrete and steel can be calculated by transforming the section, the steel being assumed to be equal to concrete, having n times the area of the steel, and acting with its center of gravity in the same line.

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$$A_t = b.h + n(A + A') \tag{39}$$

$$y = \frac{h/2 + n.p.d + n.p'.d'}{1 + n.p + n.p'} \tag{40}$$

$$I_c = \tfrac{1}{3}[y^3 + (h - y)^3]b \tag{41}$$

$$I_s = A(d - y)^2 + A'(y - d')^2 \tag{42}$$

$$I_t = I_c + n.I_s \tag{43}$$

If the reinforcement is symmetrical and equal, $y = h/2$, and $I_c = \frac{1}{12}b.h^3$, and $I_s = 2A(\frac{1}{2}h - d')^2$.

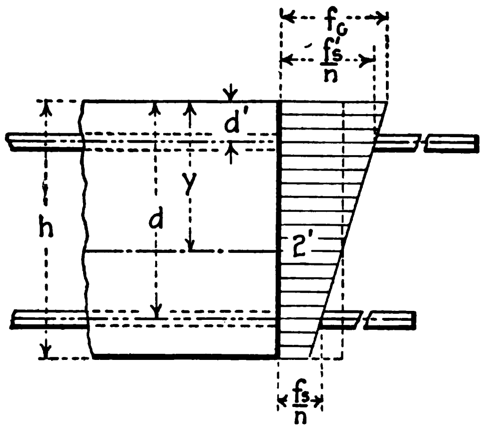

Fig. 5.

Now in Fig. 5 the direct unit stress in the concrete will be N/A, and the maximum flexural unit stress in the concrete is $\frac{M.y}{I_t}$, and the combined stresses are

$$f_c = \frac{N}{A_t} + \frac{M.y}{I_t} \tag{44}$$

$$f'_s = n\frac{N}{A_t} + \frac{n.M(y - d')}{I_t} \tag{45}$$

$$f_s = n\frac{N}{A_t} - \frac{n.M(d - y)}{I_t} \tag{46}$$

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(*b*) The stresses may be calculated directly from Fig. 6 without using the transformed sections From Fig. 6

$$f_s' = n.f_c(1 - d'/k.h) \tag{47}$$

$$f_s = n.f_c(1 - d/k.h) \tag{48}$$

$$f_c' = f_c(1 - 1/k) \tag{49}$$

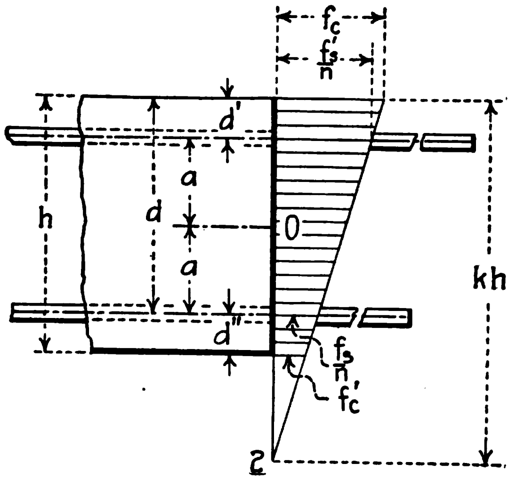

FIG. 6.

Now since the resultant normal stress equals N, we have

$$N = \tfrac{1}{2}(f_c + f_c')b.h + f_s'.A' + f_s.A \tag{50}$$

and since M = moment of all forces about the neutral axis

$$M = \tfrac{1}{2}(f_c + f'_c)b.h\frac{h}{6(2k-1)} + f'_s.A'\left(\frac{h}{2} - d'\right) - f_s.A\left(\frac{h}{2} - d\right) \tag{51}$$

The unit stresses may be calculated by means of the formulas above.

If the reinforcement is symmetrical, and $A = A'$, k is given by the equation

$$12k(1 + 2n.p)e/h = 1 + 24n.p.a^2/h^2 + 6(1 + 2n.p)e/h \tag{52}$$

and

$$M = f_c.b.h^2(1 + 24n.p.a^2/h^2)/12k \tag{53}$$

If $e/h = \frac{1}{10}$ and $p = 1.0$ per cent, $k = 2.07$.

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Case II. Stresses, Both Tension and Compression.—(*a*) If the tension as calculated by the formula $f'_s = \frac{N}{A} - \frac{M.y}{I_t}$ does not exceed, say 60 lbs. per sq. in it will be sufficient to use the formulas of Case I.

(*b*) If the tensile stresses in the concrete are too large to be neglected the stresses may be calculated as follows:

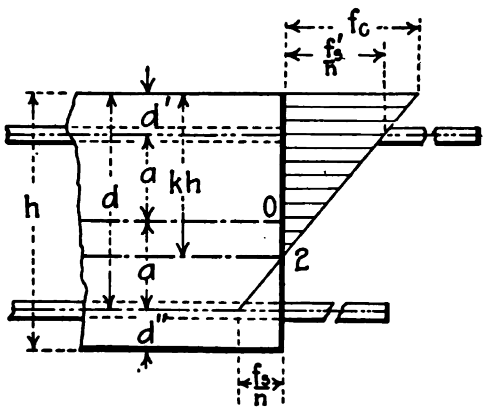

FIG. 7.

From Fig. 7 we have

$$f_s = n.f_c\left(\frac{d}{k.h} - 1\right) \tag{54}$$

and

$$f'_s = n.f_c\left(1 - \frac{d'}{k.h}\right) \tag{55}$$

The resultant fiber stress may be obtained from

$$N = \tfrac{1}{2}f_c.b.k.h + f_s'.A' - f_s.A \tag{56}$$

The moment of the fiber stresses about the horizontal axis through O is M, and

$$M = \tfrac{1}{2}f_c.b.k.h\left(\frac{h}{2} - \frac{k.h}{3}\right) + f'_s.A'\left(\frac{h}{2} - d'\right) + f_s.A\left(d - \frac{h}{2}\right) \tag{57}$$

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If the reinforcement is equal on both sides and symmetrical we have

$$k^3 - 3\left(\frac{1}{2} - \frac{e}{h}\right)k^2 + 12n.p\frac{e}{h}k = 6n.p\left(\frac{e}{h} + 2\frac{a^2}{k^2}\right) \qquad (58)$$

The greatest compression in the fiber is then obtained from

$$M = f_c.b.k^2\left[\tfrac{1}{12}k(3-2k) + \frac{2p.n}{k}\frac{a^2}{k^2}\right] \qquad (59)$$

5. **COLUMNS.** For short columns the ratio of length to least width not exceeding 15,

$$f_s = n.f_c \qquad (60)$$

$$P' = f_c.A_c + f_s.A_s \qquad (61)$$

$$= f_c.A_c[1 + (n-1)p] \qquad (62)$$

$$\frac{P'}{P} = 1 + (n-1)p \qquad (63)$$

French Commission's formula for hooped columns:

$$P' = f_c.A(1 + 15p + 32p') \qquad (64)$$

For long columns:

$$f = \frac{f_c[1 + (n-1)p]}{1 + \frac{1}{20,000}\left(\frac{l}{r}\right)^2} \qquad (65)$$

Where f is the average unit stress on the column, and l and r are the length and radius of gyration of the column respectively, both measured in the same units.

Bond or Resistance to Slipping of Reinforcing Bars.—where there is no web reinforcement the shear is taken by the concrete and the shear increments are transferred to the bars by the adhesion of the concrete to the bars. The solution is the same as that for finding the pitch of rivets in the flanges of a plate girder.

Now in (b), Fig. 1, take two right sections at a distance dx apart. Equilibrium of these two sections is maintained by the resisting moment of the bond which is equal and opposite to the moment of the vertical shear, a couple with an arm dx.

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Taking moments about center of gravity of compressive forces we have

$$V.dx = o.u.dx.j.d \tag{66}$$

where o = surface of bar for one inch in length and Σo = surface of all the bars one inch in length, u = bond developed per square inch of surface of bar, and V is the vertical shear in the beam.

Solving for u, we have

$$u = \frac{V}{j.d.\Sigma o} \tag{67}$$

Equation (67) applies to the case of horizontal bars. For inclined bars, $j.d$ will be a variable and u will be the horizontal component of the bond resistance.

Vertical and Horizontal Shearing Stresses.—At any point in a beam the vertical unit shearing stress is equal to the horizontal unit shearing stress. The horizontal shearing stress transmits the increments of tension to the reinforcing bars by bond stresses, as explained in the preceding discussion.

The amount of this horizontal stress transmitted to the reinforcing bars is by equation (67)

$$\Sigma o.u = \frac{V}{j.d}$$

Now if the horizontal shear just above the plane of the bars is v, the total horizontal shearing stress will be $v.b$, which equals $\Sigma o.u$, and

$$v = \frac{V}{b.j.d} \tag{68}$$

As an approximate formula j may be taken equal to $\frac{7}{8}$, and

$$v = {}^{8}\!/_{7}\frac{V}{b.d}$$

As no tension is assumed to exist in the concrete, the horizontal shear will be constant up to the neutral axis, above which point it decreases to zero at the top of the beam. It will be seen that lean or poor con-

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crete lacking in shearing strengtn snould not be placed below the neutral axis of beams with the idea that it may be satisfactory for the reason that the concrete is assumed to take no tension.

The same formulas apply to beams reinforced for compression as regards shear and bond stress on tensile steel.

For T-Beams.

$$u = \frac{\ }{j.d.\Sigma o} \tag{69}$$

$$v = \frac{V}{b'.j.d} \tag{70}$$

Diagonal Tension in Concrete.—In Mechanics of Materials (Merriman's Mechanics of Materials, p. 265, 1905 edition) it is shown that shear and tensile stresses combine to cause diagonal tensile stresses.

$$t = \tfrac{1}{2}s + \sqrt{\tfrac{1}{4}s^2 + v^2} \tag{71}$$

where t is the diagonal tensile unit stress, s is the horizontal tensile unit stress, and v is the horizontal or vertical shearing unit stress. The direction that stress t makes with the horizontal is one-half the angle whose cotangent is $\frac{1}{2}s/v$. If there is no tension in the concrete this reduces to

$$t = v \tag{72}$$

and t makes an angle of 45° with the horizontal.

Stresses due to diagonal tension may be carried (1) by bending the reinforced bars, or by strips sheared from them, into a diagonal position, or (2) by means of stirrups to take the vertical component of the diagonal tension, or (3) by both bent-up bars and stirrups.

Stresses in Stirrups.—The following analysis is approximate but gives results that agree closely with experiments. From formula (72) it will be seen that for no tension below the neutral axis the diagonal tension will make an angle of 45° with the horizontal; the plane of failure will then be normal to the diagonal tension and will also make an angle of 45° with the horizontal. Let V be the shear in the beam

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not carried by the concrete. Also assume that the shear is uniform over the cross-section. Then $v' = V/b.d = t$.

From Fig. 8, if s is the spacing of the vertical stirrups the stress in one stirrup is

$$P = v'.b.s = \frac{V}{d}.s \tag{73}$$

Stirrups inclined at an angle of 45° will carry the diagonal tension on a section $s.b.\cos 45°$. Then for diagonal stirrups

$$P = v'.b.s \cos 45° = 0.7 \frac{V}{d}.s \tag{74}$$

To be effective the stirrups should be spaced so that at least one stirrup will intersect the line of rupture (45° line) below the center of the

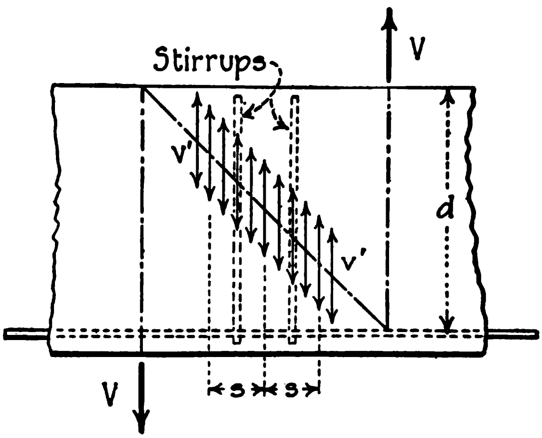

FIG. 8.

beam, which requires that s never be greater than $d/2$. Rods spaced farther apart than d are of no value.

Inclined stirrups should be rigidly fastened to the horizontal reinforcement, and all stirrups should pass around the horizontal reinforcement, and have hooked ends at the top.

To calculate the stress in a rod bent up at 45° in a beam with a uniform load the method shown in Fig. 9 may be used. The average shear

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at the support, $v' = V/b.d$, is laid off as shown. The shear that may be carried by the concrete, $v = 40$ lbs., or $v' = 35$ lbs., is subtracted. The stress in the bent-up rod is then equal to area 1-2-3-4 $\times\ b$. If the bar is bent-up at some angle other than 45° the shear v' should be laid off parallel to the bent-up bar. If stirrups are used the stress carried by them should be subtracted before calculating the stresses in the bent-up rods.

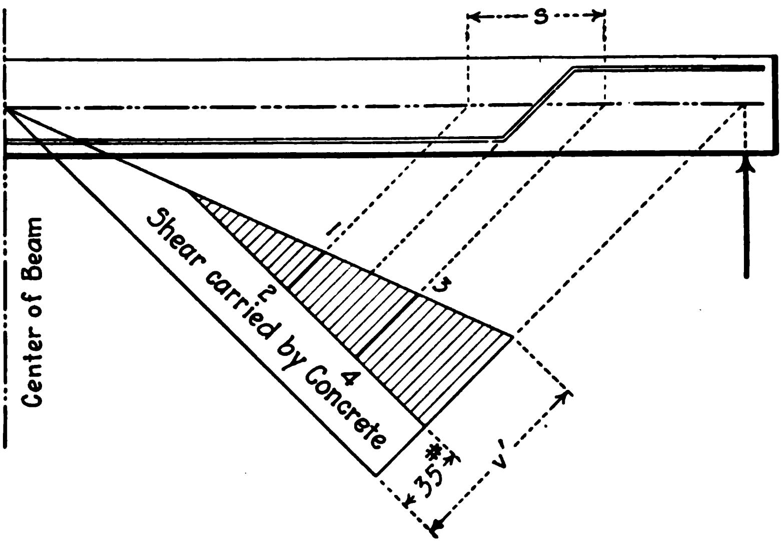

FIG. 9.

Spacing of Bars.—The lateral spacing of parallel bars should not be less than 2½ diameters, center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than 2 diameters. The clear spacing between two layers of bars should not be less than ½ inch, but the distance center to center of bars in the different layers should not be less than 2½ diameters.

T-Beams.—In beam and slab construction, an effective bond should be provided at the junction of the beam and the slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used extending over the beam and well into the slab.

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Where the slab and beam are well bonded the slab may be considered as part of the beam, but the effective width should not exceed ¼ of the span length of the beam, or its overhanging width on either side of the web should not exceed four times the thickness of the slab. Unless an efficient mechanical bond is provided the beam and slab should be cast in one operation. In the design of continuous T-beams due consideration should be given to the compressive stresses at the supports.

Floor Slabs.—Floor slabs should in general be made continuous over the supports. Square slabs should be reinforced in both directions with each system of reinforcement designed to take one-half of the load. For rectangular slabs reinforced in both directions the proportion of the load taken by the transverse reinforcement should be taken as equal to the quotient obtained by dividing the fourth power of the length by the sum of the fourth power of the length and the fourth power of the breadth, and the longitudinal reinforcement designed to take the remainder of the load. The loads carried to beams by slabs which are reinforced in two directions may be assumed to vary in accordance with the ordinates of the triangle formed by the main diagonals of the rectangle, and the moments in the beams calculated accordingly.

Bending Moments.—When the beam or slab is continuous over its supports, reinforcement should be fully provided at points of negative moment. In computing the bending moments in beams and slabs due to uniformly distributed loads the following rules should be used:

(*a*) For floor slabs the bending moments at center and supports may be taken as $w.l^2/12$ for slabs continuous over the supports; $w.l^2/10$ for slabs with one end continuous and the other end supported; and $w.l^2/8$ for slabs supported at the ends, for both dead and live loads, where w represents the loads per linear foot and l the length of the span.

(*b*) For beams the bending moment at center and supports for interior spans may be taken at $w.l^2/12$; and for end spans may be taken at $w.l^2/10$ for the center and adjoining supports for both dead and live loads. In the case of floor slabs, beams and girders designed as above

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the reinforcing steel should be rigidly fastened at the ends, or the bending moments may be taken as $w.l^2/8$. Where beams are reinforced on the compression side, the steel may be assumed to carry its proportion of compressive stress. In the case of continuous beams, tensile and compressive reinforcement over supports should extend sufficiently beyond the support to develop the required bond strength, or the bars should be bent around the flanges of beams or other frame work with bends not less than 6 inches long.

For a more complete discussion of the design of reinforced concrete structures, see Turneaure and Maurer's "Principles of Reinforced Concrete Construction."

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CHAPTER III.

Specifications for Plain and Reinforced Concrete and Steel Reinforcement.

Concrete Materials.

1. **Cement.**—The cement shall be Portland and shall meet the requirements of the standard specifications of the American Society for Testing Materials.

2. **Fine Aggregates.**—Fine aggregate shall consist of sand, crushed stone, or gravel screenings graded from fine to coarse, and passing when dry a screen having ¼ in. diameter holes; it shall preferably be of siliceous material, clean. coarse, free from vegetable loam or other deleterious matter, and not more than 6 per cent shall pass a sieve having 100 meshes per linear inch.

3. **Strength of Mortar.**—Mortars composed of one part Portland cement and three parts fine aggregate by weight when made into briquettes shall show a tensile strength of at least equal to 70 per cent of the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand.

4. **Coarse Aggregates.**—Coarse aggregate shall consist of crushed stone or gravel, graded in size, and which is retained on a screen having ¼ in. diameter holes; it shall be clean, hard, durable, and free from all deleterious material. Aggregates containing soft, flat or elongated particles shall not be used.

5. **Water.**—The water used in mixing concrete shall be free from oil, acid and injurious amounts of alkalies or vegetable matter.

Steel Reinforcement.

6. **Manufacture.**—Steel shall be made by the open-hearth process. Rerolled material will not be accepted.

7. Plates and shapes used for reinforcement shall be of structural steel only. Bars and wire may be of structural steel or high carbon steel.

8. **Schedule of Requirements.**—The chemical and physical properties shall conform to the following limits:

Elements Considered.	Structural Steel.	High Carbon Steel.
Phosphorus, max... Basic.........	0.04 per cent	0.04 per cent
Phosphorus, max... Acid.........	0.06 per cent	0.06 per cent
Sulphur, maximum....................	0.05 per cent	0.05 per cent
Ultimate tensile strength.	Desired.	Desired
Pounds per square inch...............	60,000	88,000
Elong., min. % in 8″, Fig. 1...	$\frac{1,500,000^{*}}{\text{Ult. tensile strength}}$	$\frac{1,000,000}{\text{Ult. tensile strength}}$
Character of fracture..................	Silky	Silky or finely granular.
Cold Bends without Fracture......	180° flat †	180° $d = 4t$.‡

* See paragraph 15. † See paragraphs 16 and 17. ‡ "d = 4t" signifies "around a pin whose diameter is four times the thickness of the specimen."

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9. **Yield Point.**—The yield point for bars and wire, as indicated by the drop of the beam, shall be not less than 60 per cent of the ultimate tensile strength.

10. **Allowable Variations.**—If the ultimate strength varies more than 4,000 lbs. for structural steel or 6,000 lbs. for high carbon steel, a retest shall be made on the same gage, which, to be acceptable, shall be within 5,000 lbs., for structural steel, or 8,000 lbs. for high carbon steel, of the desired ultimate.

11. **Chemical Analyses.**—Chemical determinations of the percentages of carbon, phosphorus, sulphur and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector.

12. **Form of Specimens. Plates, Shapes and Bars.**—Specimens for tensile and bending tests for plates and shapes shall be made by cutting coupons from the finished product, which shall have both faces rolled and both edges milled to the form shown by Fig. 1; or with both edges parallel; or they may be turned to a diameter of ½ in. with enlarged ends, Fig. 2.

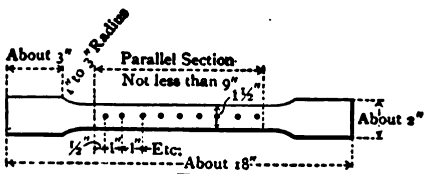

FIG. 1.

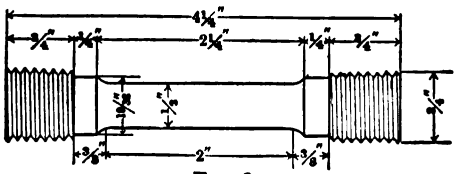

FIG. 2.

13. Bars shall be tested in their finished form.

14. **Number of Tests.**—At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing ⅜ in. and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

15. **Modifications in Elongation.**—For material less than $\frac{5}{16}$ in. and more than ¾ in. in thickness the following modifications will be allowed in the requirements for elongation:

(a) For each $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in. a deduction of 2½ will be allowed from the specified percentage.

(b) For each ⅛ in. in thickness above ¾ in., a deduction of 1 will be allowed from the specified percentage.

16. **Bending Tests.**—Bending tests may be made by pressure or by blows. Shapes and bars less than one inch thick shall bend as called for in paragraph 8.

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17. **Thick Material.**—Test specimens one inch thick and over shall bend cold 180 degrees around a pin, the diameter of which, for structural steel, is twice the thickness of the specimen, and for high carbon steel, is six times the thickness of the specimen, without fracture on the outside of the bend.

18. **Finish.**—Finished material shall be free from injurious seams, flaws, cracks, defective edges or other defects, and have a smooth, uniform and workmanlike finish.

19. **Stamping.**—Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it, except that bar steel and other small parts may be bundled with the above marks on an attached metal tag.

20. **Defective Material.**—Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious defects, will be rejected and shall be replaced by the manufacturer at his own cost.

21. All reinforcing steel shall be free from excessive rust, loose scale, or other coatings of any character, which would reduce or destroy the bond.

Workmanship.

22. **Unit Measure.**—The unit of measure shall be the cubic foot. A barrel containing 4 bags weighing not less than 94 lbs. each shall be assumed as containing 3.8 cubic feet of cement. Fine and coarse aggregates shall be measured separately as loosely thrown into the measuring receptacle.

23. **Relation of Fine and Coarse Aggregates.**—The fine and coarse aggregates shall be used in such relative proportions as will insure maximum density.

24. **Proportions.**—The proportions of materials for the different classes of concrete shall be as follows:

Class	Use.	Cement.	Aggregates.	
			Fine.	Coarse.

25. **Proportions.**—For plain concrete, a proportion of 1:9 (unless otherwise specified) shall be used, i. e., one part of cement to a total of nine parts of fine and coarse aggregates *measured separately;* for example, 1 cement, 3 fine aggregate, 6 coarse aggregate.

26. For reinforced concrete a proportion of 1:6 (unless otherwise specified) shall be used, i. e., one part of cement to a total of six parts of fine and coarse aggregates *measured separately.*

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27. **Mixing.**—The ingredients of concrete shall be thoroughly mixed to the desired consistency, and the mixing shall continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous.

28. **Measuring Proportions.**—The various ingredients, including the water, shall be measured separately, and the methods of measurement shall be such as to secure the proper proportions at all times.

29. **Machine Mixing.**—A machine mixer, preferably of the batch type, shall be used wherever the volume of the work will justify the expense of installing the plant. The requirements demanded are that the product delivered shall be of the specified proportions and consistency and thoroughly mixed.

30. **Hand Mixing.**—When it is necessary to mix by hand, the mixing shall be on a watertight platform of sufficient size to accommodate men and materials for the progressive and rapid mixing of at least two batches of concrete at the same time. Batches shall not exceed one-half cubic yard each. The mixing shall be done as follows: The fine aggregate shall be spread evenly upon the platform, then the cement upon the fine aggregates, and these mixed thoroughly until of an even color. The coarse aggregates which, if dry, shall first be thoroughly wetted down shall then be added to the mixture. The mass shall then be turned with shovels until thoroughly mixed and all of the aggregate covered with mortar, water being added as the mixing proceeds. Or, at the option of the engineer, the coarse aggregate may be added after, instead of before, adding the water.

31. **Consistency.**—The materials shall be mixed wet enough to produce a concrete of such consistency that it will flow into the forms and about the metal reinforcement, and which, on the other hand, can be conveyed from the place of mixing to the forms without separation of the coarse aggregate from the mortar.

32. **Retempering.**—Retempering mortar or concrete, i. e., remixing with water after it has partially set, will not be permitted.

33. **Placing of Concrete.**—Concrete after the addition of water to the mix shall be handled rapidly from the place of mixing to the place of final deposit, and under no circumstances shall concrete be used that has partially set before final placing.

34. The concrete shall be deposited in such a manner as will prevent the separation of the ingredients and permit the most thorough compacting. It shall be compacted by working with a straight shovel or slicing tool kept moving up and down until all the ingredients have settled in their proper place and the surplus water is forced to the surface. In general, except in arch work, all concrete should be deposited in horizontal layers of uniform thickness throughout.

35. In depositing concrete under water, special care shall be exercised to prevent the cement from floating away, and to prevent the formation of laitance.

36. Before depositing concrete in forms, the forms shall be thoroughly wetted except in freezing weather, and the space to be occupied by the concrete cleared of debris.

37. Before placing new concrete on or against concrete which has set, the

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surface of the latter shall be roughened, thoroughly cleansed of foreign material and laitance, drenched and slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate.

38. The faces of concrete exposed to premature drying shall be kept wet for a period of at least three days.

39. **Freezing Weather.**—The concrete shall not be mixed or deposited at a freezing temperature, unless special precautions, approved by the engineer, are taken to avoid the use of materials containing frost or covered with ice crystals, and to provide means to prevent the concrete from freezing.

When the temperature of the air is below 40° F. during the time of mixing and placing concrete, the water used in mixing concrete shall be heated to such a temperature that the temperature of the concrete mixture shall not be less than 60° when it reaches its final position in the forms. Care shall be used that the cement shall not be injured by boiling water.

40. **Rubble Concrete.**—Where the concrete is to be deposited in massive work, clean, large stones, evenly distributed, thoroughly bedded and entirely surrounded by concrete, may be used, at the option of the engineer.

41. **Forms.**—Forms shall be substantial and unyielding and built so that the concrete shall conform to the designed dimensions and contours, and so constructed as to prevent the leakage of mortar.

42. **Removing Forms.**—The forms shall not be removed until authorized by the engineer.

43. **Form Lumber.**—For all important work, the lumber used for face work shall be dressed to a uniform thickness and width; shall be sound and free from loose knots and secured to the studding or uprights in horizontal lines.

44. For backings and other rough work undressed lumber may be used.

45. Where corners of the masonry and other projections liable to injury occur, suitable moldings shall be placed in the angles of the forms to round or bevel them off.

46. Lumber once used in forms shall be cleaned before being used again.

Details of Construction.

47. **Splicing Reinforcement.**—Wherever it is necessary to splice the reinforcement otherwise than as shown on the plans, the character of the splice shall be decided by the engineer on the basis of the safe bond stress and the stress in the reinforcement at the point of splice. Splices shall not be made at points of maximum stress. The reinforcement shall be carefully placed in accordance with the plans, and adequate means shall be provided to hold it in its proper position until the concrete has been deposited and compacted.

48. **Joints in Concrete.**—Concrete structures, wherever possible, shall be cast at one operation, but when this is not possible, the resulting joint shall be formed where it will least impair the strength and appearance of the structure.

49. Girders and slabs shall not be constructed over freshly formed walls or columns without permitting a period of at least four hours to elapse to provide for settlement or shrinkage in the supports. Before resuming work, the tops of such walls or columns shall be cleaned of foreign matter and laitance.

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50. A triangular-shaped groove shall be formed at the surface of the concrete at vertical joints in walls and abutments.

51. **Surface Finish.**—Except where a special finish is required, a spade or special tool shall always be worked between the concrete and the form to force back the coarse aggregates and produce a mortar face.

52. **Top Surfaces.**—Top surfaces shall generally be "struck" with a straight edge or "floated" after the coarse aggregates have been forced below the surface.

53. **Sidewalk Finish.**—Where a "sidewalk finish" is called for on the plans, it shall be made by spreading a layer of 1 : 2 mortar at least ¾ in. thick, troweling the same to a smooth surface. This finishing coat shall be put on before the concrete has taken its initial set.

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